
Journal of the
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Journal of the
HYDRAULICS DIVISION
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EARLY HISTORY OF HYDROMETRY IN THE UNITED STATES

By Steponas Kolupaila,¹

SYNOPSIS

The history of hydrometry (stream flow measurements) is very enlightening and inspiring. Field methods which were established in America several decades ago, were widely adapted as standards in many other countries. The names of American pioneers, such as Humphreys and Abbot, Francis, Herschel, Hoyt and Grover, Stout, Stevens, Horton, Allen and Gibson, are well known throughout the world. However, many distinguished and deserving men have been almost forgotten in their own country.

The aim of this study is to recall the deeds of the American pioneers in our field of science and engineering, examine their achievements, give credit where it is due, and in this way to pay tribute to our glorious past.

Motto: A nation which forgets its past, loses its future (Sir Arthur Bryant)

Note.—Discussion open until June 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 1, January, 1960.

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FLOW MEASUREMENTS IN EARLY AMERICA

The early settlers of American territory used rivers extensively for transportation, water supply, and power. By the 19th century thousands of water mills were in operation. A unit of water discharge in use in New England, called the "Millwright's inch of water", is identified as the area of contracted vein, in square inches. This area was assumed as 62% of the area of the orifice. Curiously, neither the water head on the orifice, nor the velocity of the jet were taken into consideration.

Some stream measurements must have been made for those installations. The results are unknown, because they were not preserved. Only a few early high-water marks can be found on river banks. The value of river-stage observations and need for their preservation was still not understood at that time.

Originally the most important task of hydraulic engineering was the construction and improvement of waterways. George Washington promoted a project to connect the headwaters of the Ohio River with the sea via the Potomac River. Washington, a land surveyor, investigated the selected route and made the necessary topographic and hydrometric measurements.² This projected canal was built in 1784. Two ship locks were installed, the first known in America. At the time of construction Washington was Commander in Chief of the American Army.

The first federal institution for the improvement of harbors and navigable rivers was the Corps of Engineers, U. S. Army, organized in 1802. Occasional measurements were made by the Engineers for temporary purposes in planning projects. Usually, however, flow records were not continued and not published. Therefore little is known about early hydrometric measurements.

ERA OF THE GREAT NAVIGATION CANALS

The first half of the nineteenth century was a period of intensive planning and construction of artificial waterways.

C. E. Sherman reproduced an old report by D. S. Bates of his investigations made in 1824.³ That report was submitted to the Board of Canal Commissioners, which had been appointed to study projects of a navigation canal between the Ohio River and Lake Erie. After measuring river flow and estimating the runoff at more than forty locations in central Ohio, Bates recommended the Board condemn those investigated routes, since the available water was not sufficient to feed the navigation canal at the summit. Bates used weirs with rectangular notches on smaller creeks and floats on larger streams. His results, obtained during a drought in 1824, were substantially lower than the low-flow records of 1925 and 1930. This difference may be explained by the fact that a century ago almost the entire drainage area was covered by forest. Sherman states that "judged by the care with which the work was done, Bates' measurements are entitled to great credit".

In 1823 Samuel Forrer, Ohio State Engineer, investigated another route, through the Scioto and Sandusky Rivers. Together with a Commissioner

² Ch. C. Long, George Washington, the engineer. *The Military Engineer*, 31 (1939), No. 177, May-June, pp. 172-173. Washington.

³ C. E. Sherman, Ohio stream flow. *Ohio State University Engineering Experiment Station, Bulletin No. 73*. Columbus, Ohio, 1932.

Alfred Kelley they measured the discharge of the Sandusky River in August 1823, and several other rivers in September. Two weirs were used, 18 in. wide. These were probably the first discharge measurements, of which there are records.⁴

The method of weir measurement was described in a contemporary periodical "Civil Engineer and Herald of Internal Improvement", Columbus, Ohio, 1828.

One of the greatest engineering enterprises of that period was the construction of the Erie Canal between Lake Erie and the Hudson River. The original canal was 363 miles long, 40 ft wide and 4 ft deep. It had 83 locks, 90 ft by 15 ft each. The total head of lockage was 680 ft. The waterway was to accommodate boats up to 75 tons. Constructed between 1817 and 1825 at a cost of \$7,000,000, the waterway was opened in November 1825 with impressive ceremonies. This canal was "the first American school of engineering" and had an enormous effect on the growth of New York City and on the technical progress in the United States. Three "hydrostatic locks" were established on hydrometric devices for the measurement of the ship displacement. They were used for the purpose of toll assessment, which was determined by the cargo weight. These locks proved to be "wonderfully efficient".^{5,6}

Evidently, some gagings were made at that time, although their records were not preserved since the water supply for the original Erie canal was greatly in excess of the amount required. During the construction of the canal it was necessary to determine the water loss from the canal by seepage and evaporation. In 1824 J. B. Jervis, one of the Erie canal engineers, made such measurements in the eastern division, while Bates made them in the western division. The flow of all feeders was measured for that purpose. Jervis determined the loss of 2.1 cfs per mile, while Bates obtained 1.7 cfs per mile. This smaller value was later confirmed by H. Tracy and S. Talcott, in 1839.

Jervis established the earliest continuous observations of the river flow in 1835, when he was chief engineer of the Chenango canal. The first permanent gaging stations were located in Eaton and Madison brooks, in Madison County,



FIG. 1.—SAMUEL FORRER

⁴ A. H. Frazier, First streamflow measurement in the United States. 1958.

⁵ A. F. Harlow, Old towpaths. The story of the American canal era. New York & London 1920, D. Appleton & Co.

⁶ N. W. Whitford, History of the canal system of the State of New York. Albany 1906, 2 volumes.

N. Y. The data of 1835 is assumed as the start of systematic river investigations in the United States. The City of New York has continuous records of Croton River since 1868.⁷

WATER POWER FOR INDUSTRY

The progress in water measurements in New England is associated with the development of textile industry in Lowell, Mass.⁸:

"At its zenith it was one of the marvels of the day. It was shown with pride as the epitome of all that an industrial city should be, the Venice of America, with its "mile of mills" along the river, its boarding houses for the operatives, and its churches and public buildings".

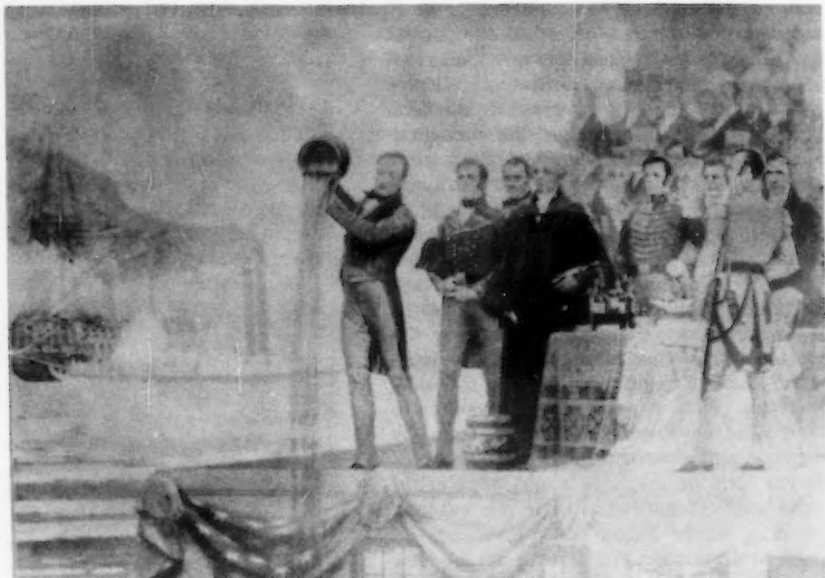


FIG. 2.—ERIE CANAL DEDICATION, 1824

The whole industry in Lowell was operated by water power. Since 1823 large breast wheels moved all machinery by direct mechanical transmission. Water was supplied from the Merrimac River through the Pawtucket navigation canal. A company of "The Proprietors of the Locks and Canals on Merrimac River", chartered in 1792, had controlled and distributed water among 40 textile mills. The water was sold to the mills by the "Mill power", which meant 25 cfs on a fall of 30 ft, equal to about 60 hp. The price of 300 dollars per mill power per year was established "forever". It has not changed since 1853.

⁷ R. E. Horton, Report of Bureau of Hydraulics, State of New York. Albany 1911, p. 305.

⁸ J. G. W. Thomas, The Proprietors of the locks and canals on Merrimac River, Journal of the Boston Society of Civil Engineers, 44(1957), No. 2, Apr., pp. 61-69. Boston.

In 1830 Ithamar A. Beard constructed a gage-wheel with paddles and installed it across the canal carrying water for Lowell's industry. A huge installation for continuous water measurement was designed by three skillful engineers, James F. Baldwin, George W. Whistler and Charles S. Storrow, in 1841. The quantity of water drawn from the Merrimac River was measured in a canal about 29 ft wide and 8 ft deep. A large gage-wheel, 16 ft in diameter, consisting of seven coupled wheels, each 10 ft wide, with a supporting pier between each wheel, was assembled across the channel on a horizontal shaft. The channel at that site was enlarged to 80 ft, and the depth reduced to 4.5 ft. A gap of 1/4 in. was left between the wheel and the apron at the bottom, as well as between the pier surfaces. A series of 24 paddles were attached radially

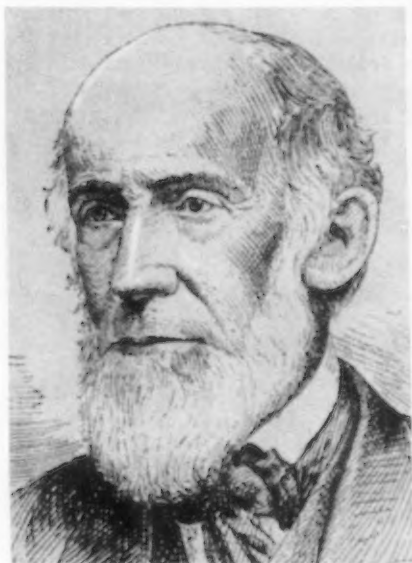


FIG. 3.—J. B. JERVIS

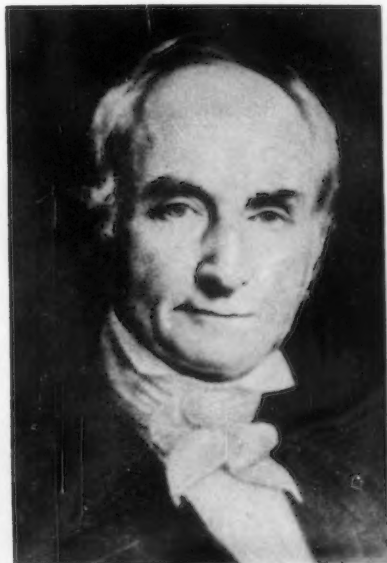


FIG. 4.—JAMES F. BALDWIN

around the wheels, their lower part immersed. Thus the water pushed the paddles and rotated the gage-wheel. The volume of water entrapped between the paddles was determined by computation. The number of revolutions of the wheel was counted by a clock-work, connected to one end of the shaft. The velocity of water in the channel was 2 fps to 3.5 fps, the water discharge 400 cfs to 600 cfs.⁹ By its dimensions this gage-wheel was probably the largest water meter ever built.¹⁰

⁹ F. Van Winkle, Stream flow at single cross-section. *Power and the Engineer*, 32 (1910), part 2, Aug. 30, pp. 1553-1557. New York.

¹⁰ The first gage-wheel, a "molinetto," was constructed by an Italian Francesco Domenico Michelotti in 1767. (G. Masetti, *Descrizione, esame e teoria di tutti tachimetri idraulici fino ad ora conosciuti*. Bologna 1824). The same idea was applied by an Australian engineer J. S. Dethridge for measuring irrigation water in 1913. (An Australian water meter for irrigation supplies. *Engineering News*, 70(1913), No. 26, Dec. 25, p. 1283. New York).

EARLY RIVER INVESTIGATIONS

The aim of the first travelers along American rivers was discovery and exploration of the natural ways of communication and trade.

In 1804 Thomas Jefferson ordered the first expedition across the country. Two officers, Meriwether Lewis and William Clark, with a party of 46 soldiers went up the Missouri River by boat and after crossing the Divide continued their travel by canoe along the Ourigan (Columbia) River to the Pacific. They returned in 1806.

In 1821 Simon Bernard and Joseph Totten led an expedition by boat along the Ohio and Lower Mississippi rivers for navigation purposes. In a report¹¹ they described twenty-one falls of the Ohio River and sand bars between the



FIG. 5.—GEORGE W. WHISTLER



FIG. 6.—CHARLES S. STORROW

falls, as obstacles to navigation. Their comments on the Mississippi River were of a more general character. In 1836-39 Jean Nicolas Nicollet explored the Upper Mississippi River.

The earlier river investigations were difficult and perilous, particularly in the wild west.

Joseph C. Ives explored the lower part of the Colorado River in 1857-58 and presented an interesting report.¹² An iron ship, 50 ft long, with a steam engine,

¹¹ S. Bernard and J. Totten, Report of the Board of Engineers on the Ohio and Mississippi Rivers, from an examination made in the months of Sept., Oct., Nov., and Dec., 1821

¹² J. C. Ives, Report upon the Colorado River of the West. Washington 1861, Congressional Documents No. 1068, 131 p.

was built in a workshop in Philadelphia, Pa. The sections of it were transported by ship, over the Isthmus of Panama by railway, and again by seaway to San Francisco, then to the head of the Gulf of California. Here, at Robinson's Landing, the steamer was assembled and launched at high tide. The expedition observed the river level fluctuations on a gage and registered meteorological data. The party reached the Grand Canyon by the ship, called the "Explorer", and continued investigation by a pack-train of mules.

The dangerous and treacherous canyons of the Colorado River were explored by J. W. Powell in 1867-1878. Powell was "one of the most colorful figures on the American scene".¹³ Mostly self-educated, one-handed (as a Captain of Artillery of the Union Army, he lost his right arm in a battle during the Civil war in 1862), Professor of geology at the Illinois Wesleyan University, Bloomington, Ill., and curator of the Museum of Natural History of Illinois, he organized and led several expeditions to the Rocky Mountains and the Colorado River basin. In 1869 with a group of nine courageous men he

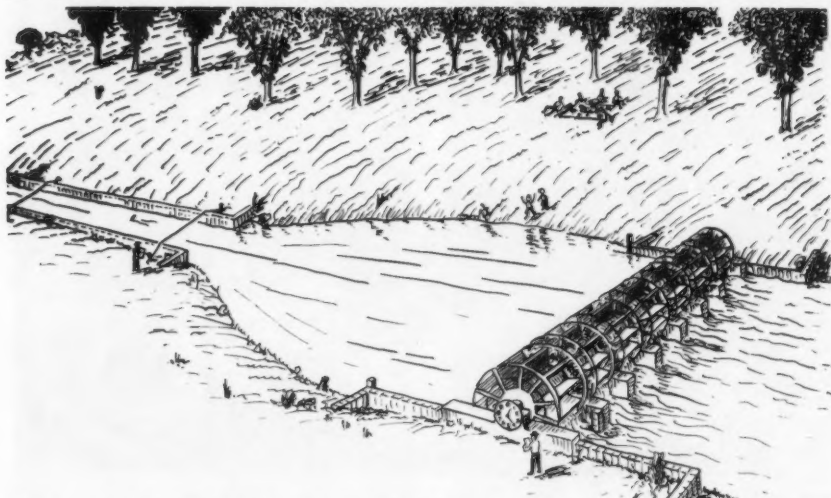


FIG. 7.—MEASURING WHEEL IN THE MERRIMAC CANAL, 1841. RECONSTRUCTION BY A. H. FRAZIER

accomplished his famous and adventurous descent in four boats through a thousand miles of canyons. The travellers were exhausted by passing dangerous rapids, their equipment sank and was lost, and food ran short. Three men, who returned earlier, were killed by Indians. The second expedition, better equipped, repeated the journey along the Colorado River in 1871. The program of this expedition included some hydrometric measurements. High water forced the party to discontinue their trip before they reached the Grand Canyon. The purpose of these explorations was to study topography, geology and hydrology of the region, for conservation of the natural resources and for possible irrigation of the arid West. Powell was the first person to anticipate the great

¹³ W. C. Darrah, Powell of the Colorado. Princeton 1951, Princeton University Press, 426 p.

storage dams on the Colorado River. His report¹⁴ was published in 1875 and has been reprinted many times in various magazines and books of adventure.

POWER OF THE NIAGARA FALLS

Zachariah Allen, a scientist of Providence, Rhode Island, deserves the credit for the first discharge measurement of a large river. In the summer of 1841 he visited the Niagara Falls and decided to determine the power of that "marvel of the world." First of all, water discharge was to be measured.

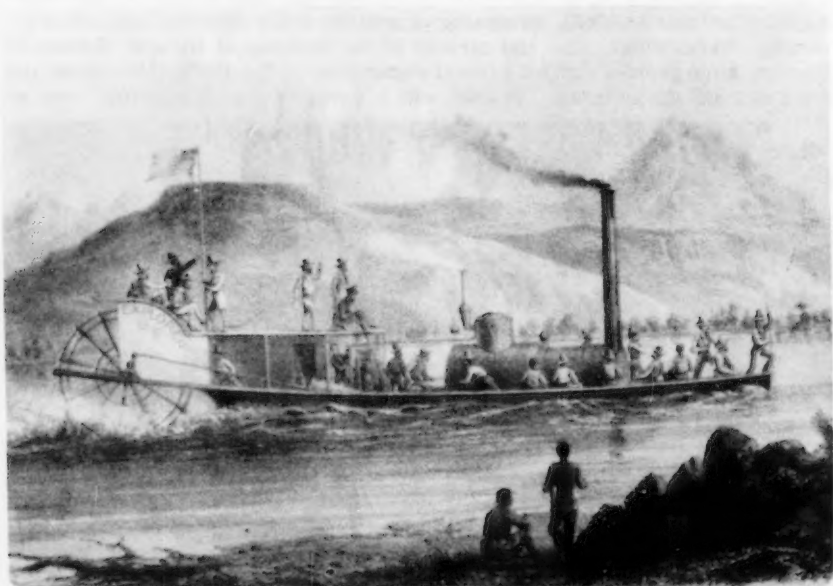


FIG. 8.—EXPLORATION OF THE COLORADO RIVER, 1858

For this reason Allen complained:¹⁵ "Very little attention appears to have been hitherto bestowed on the investigation of the comparative volumes of water discharged by the great rivers of the globe". Conditions in the Niagara River were quite favorable, since the discharge fluctuations were less significant than in other rivers. A reach was selected below the outlet from Lake Erie, where almost uniform depth of about 30 ft was found. Three cross sections were measured, 600 ft apart, and surface floats were run in 10 places.

¹⁴ J. W. Powell, *Exploration of the Colorado River of the West and its tributaries*. Washington 1875, 291 p. Abridged: Chicago 1957, University of Chicago Press, 138 p.

¹⁵ Z. Allen, On the volume of the Niagara River, as deduced from measurements made in 1841 by E. R. Blackwell, and calculated by Z. Allen. *The American Journal of Science and Arts*, 46(1844), No. 1, Oct.-Nov. 1843, pp. 67-73. New Haven.

E. R. Blackwell was in charge of these measurements. The surface velocity was as high as 11.75 fps. Following the Eytelwein formula, the average velocity on the verticals was taken as 0.90 of the surface velocity. The water discharge was computed¹⁶ as 375,000 cfs. Allen assumed the fall of 160 ft and computed the power of Niagara Falls as 6,800,000 hp, 2/3 of which, or 4,500,000 hp, could be used. This was 40 times more than the power of the whole industry of Great Britain at that time. The value of total power, rounded to 7,000,000 hp, was repeated by everybody for about 100 yr. The true average discharge of the Niagara River since 1860 has been about 203,000 cfs, the monthly minimum 117,000 cfs, and the maximum 254,000 cfs. Thus, the first measured discharge was about 85% larger than the actual average discharge, and about 50% larger than the maximum.

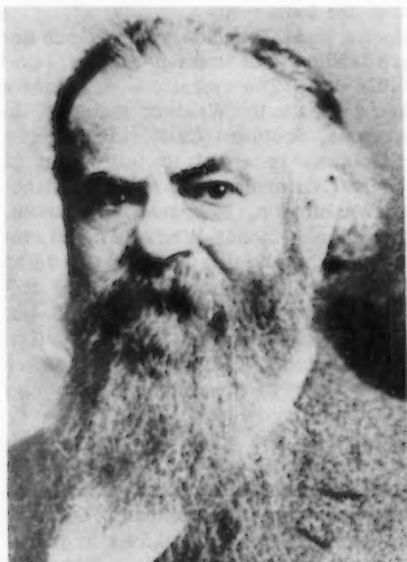


FIG. 9.—J. W. POWELL



FIG. 10.—COLORADO RIVER RAPIDS, 1869

WATER IN THE WEST

Colonization of the West was stimulated by the gold rush to Colorado and California in 1849. Increase of population created food demand, which resulted in irrigation. Water was essential in mining and in agriculture. Water in the West had literally the value of gold. Water was a condition for colonization and life. The first water measurements followed from the high price of water.

¹⁶ The writer repeated the computation by graphical method and obtained the discharge of 340,000 cubic feet per second, about 10 per cent less than by Allen.

Here the "Miner's inch" and the "Colorado inch" were the oldest units for the measurement and delivery of water. The term "inch" expressed the quantity of water discharged through a rectangular orifice of 1 sq in. area under a certain head for 24 hr. A device for the control of delivery was also called "the inch." The oldest Colorado inch, earlier known as the Max Clark's gauge, was a wooden chamber, 12 ft to 15 ft long, with a sluice-gate at the upper end and a rectangular orifice at the lower end. A horizontal slide could open the desired width of the orifice. The Colorado law¹⁷ fixed the following rule for water measurement:

Every inch of water shall be considered equal to an inch square orifice under a 5-in. pressure, and a 5-in. pressure shall be from the top of the orifice of the box put into banks of the ditch to the surface of the water, . . . and said box put into the banks of ditch shall have a descending grade from the water in the ditch of not less than 1/8 in. to the foot.



FIG. 11.—ZACHARIAH ALLEN

Such a legal Colorado miner's inch delivers 2250 cu ft of water per 24 hr, equal to 0.026 cfs. The magnitude of miner's inches varies in the Western states: 1 in. in Arizona, Southern California, Oregon and Montana is equal to 0.025 cfs; in Northern California, New Mexico, Idaho, Utah, Washington, Nebraska, S. Dakota, N. Dakota and Kansas it equals 0.020 cfs; and in British Columbia (Canada) it is 0.028 cfs. This variance resulted from the different assumed head, from 4 in. to 6-1/2 in. The use of conflicting units has led to much confusion. Today these units have disappeared from practice, they remain only in legal documents.

Many improved types of water meters were designed in the West. In 1886 A. D. Foote offered his water meter with a 4-in. high slot and a minimum head. Old devices were gradually displaced by weirs and flumes.

In old times Spanish and Mexican missions carried some hydrologic observations. Henry B. Lynch collected scattered records from various diaries on rainfall and runoff data in Southern California, and some lake-level observations, and published them in a pamphlet.¹⁸ He found that there had been no material change in the mean climatic conditions in that region in the 162 yr covered by observations.

EARLIEST GAGINGS OF THE MISSISSIPPI RIVER

The oldest hydraulic engineering work in the Lower Mississippi was the development of levees for the protection of New Orleans, La. against floods, which was begun in 1717.

¹⁷ Chapter 102, sec. 3.

¹⁸ H. B. Lynch, Rainfall and stream runoff in Southern California since 1769. Los Angeles 1931, The Metropolitan Water District of So. Cal., 31 p.

The records of river floods were originated by Winthrop Sargent at Natchez, Mississippi, in 1798, and were carried on by him up to 1810. They were continued by Samuel Davis to 1841 and by Caleb G. Forshey to 1848. Later these observations were transferred to Carrollton, La. Stage readings were made on temporary gages at certain permanent points during high water periods. Continuous daily observations were established in July, 1846.

The first efforts to measure or to estimate water discharge and the rate of sediment transport began about 1838. The pioneers in hydrometry of the Mississippi River were:

1) Andrew Talcott, noted astronomer, who investigated the Delta of the Mississippi River in July 1838;

2) John Leonard Riddell, Professor of Medicine, who made measurements at New Orleans in 1843;

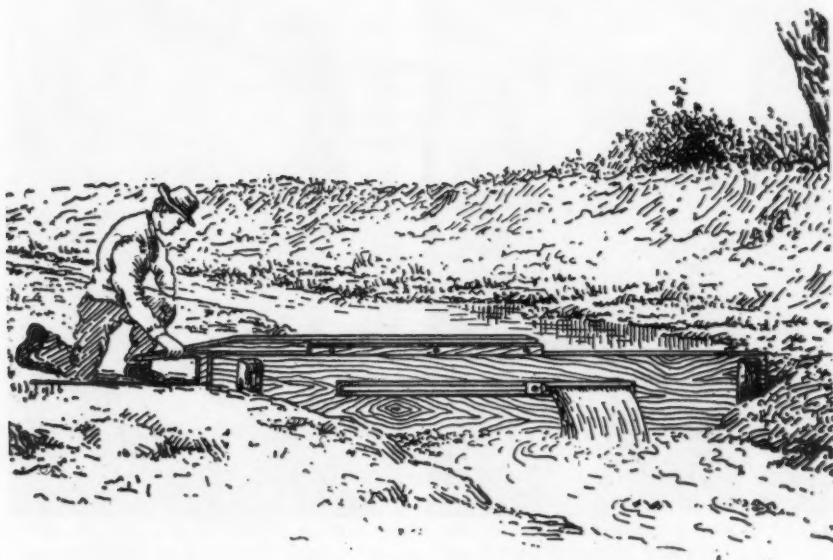


FIG. 12.—MINER'S INCH

3) Andrew Brown, who carried on observations at Natchez, Miss., from 1846 to 1848;

4) Robert A. Marr, Lieutenant, U. S. Navy, who measured the flow of the river at Memphis, Tenn., in 1849 and 1850-1851;

5) Charles Ellet, Jr., who determined the runoff of the Mississippi River below the Red River during two months of flood in 1851 and evaluated the annual runoff; and

6) Caleb G. Forshey, Professor, U. S. Military Academy, who investigated the river at different points in 1850-1853.

The measurement of discharge of a large river was extremely difficult, particularly with the simple instruments and imperfect equipment of that time.

A. A. Humphreys highly praised the pioneer work voluntarily accomplished by Forshey. In the famous Mississippi Report he pays tribute to him in these words:

"Professor Forshey is entitled to great credit for the zealous and intelligent manner in which he devoted himself, for many years . . . to observing and collecting facts relative to river phenomena, without aid from any source whatever, he thus accumulated a mass of valuable material. When it is considered how difficult and costly perfect observations are, of the character of some of those made by him as an amateur, it is a matter of surprise that so much should have been done by the unassisted enterprise of a private individual."

At that time river discharge was measured by primitive tools and means. Marr fitted a line across the river and supported it by cork-floats in order to

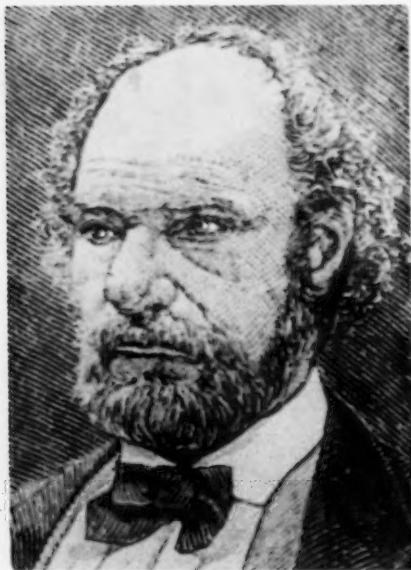


FIG. 13.—CALEB G. FORSHEY

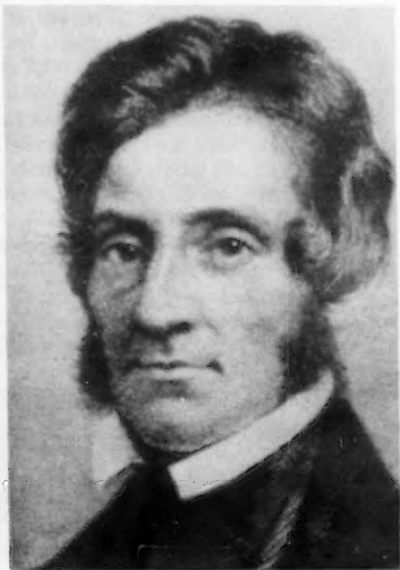


FIG. 14.—CHARLES ELLET

keep it on the surface. A weight on a string was used for depth sounding. Surface floats (wood chips) were run on a 1-mile course. The surface velocity so determined was then reduced 10% to obtain the average velocity. The river cross section was divided into three parts, the average depth of each part was multiplied by the width and by the average velocity. The sum of these three products was estimated to be the river discharge. Such measurements or evaluations were made every day from March 1850 to March 1851, and the annual runoff was computed to be 13,709,006,232,791 cu ft per yr.

Samples of water were taken daily from the surface at the middle of the river. Experiments showed that the concentration of silt increased with the depth. The sediment was equal to 1/2950 part of water flow.¹⁹

¹⁹ R. A. Marr, *Observations on the Mississippi River, at Memphis, Tenn., March 1, 1850, to March 1, 1851. Appendix to the Washington Astronomical Observations for 1847.* Washington 1853, 22 p.

The results of some of the earliest determinations or estimations of the Mississippi River flow, in cubic feet per second, are as follows:

Talcott, before Delta, July 1838	809,565
Riddell, at New Orleans, 1843	352,000
Brown, at Natchez, 1848	471,000
Marr, at Memphis, 1850	434,000
Ellet, below Red River, 1851	667,000
Forshey, above Red River, 1819-1849	389,000
in 1860 corrected to	615,000

In spite of the inadequacy of early methods, some results of these determinations are quite acceptable. The average annual discharge for the last 25 yr has been computed as follows:

Mississippi River at Memphis	466,000 cfs
at Natchez	575,000 cfs
at Red River Landing, La.	620,000 cfs

Marr estimated that only 1/1200 part of the rain-water falling in the valley of the Mississippi River reaches the Gulf. Ellet corrected this figure. He evaluated the runoff coefficient for the Lower Mississippi to be about 0.20, Forshey obtained 0.247. The presently accepted figure is about 0.240.

CHARLES ELLET AND THE OHIO RIVER

The real pioneer in hydrometry was the eminent civil engineer and hero of the Civil War, Charles Ellet, Jr. Endowed with exceptional capability and courage, educated in France, he built bridges, planned navigation canals and flood-control works. The success of his last invention, the armored ram-boat, was crowned by his dramatic death on the Mississippi River.

In 1849 Ellet constructed the suspension bridge over the Ohio River at Wheeling, W. Va. The bridge was 1010 ft long and at that time its span was the longest in the world. A storm destroyed this bridge in 1854. During construction of the bridge Ellet displayed great interest in the Ohio River. He procured data on water-stage observations, beginning in 1838, and established a discharge-measurement section in a suitable location. The cross-section at this station was divided into four parts and the surface velocity was measured in each part by properly loaded floats. The Prony formula was applied for reduction of velocity. The water discharge was computed by multiplying average velocities by partial areas and taking their sum. Ellet also developed a stage-discharge curve. The gage readings were reduced to the water depth over a downstream sand bar. An empirical equation $Q = a h^2 - b h^3$ was established, where a and b were constants, and h the reduced water height. The actual relationship was more complicated, because the water-height reduction varied with the river stage. This equation was applied for the translation of daily stages into discharges. From 1843 to 1848, the daily discharges were published. The average discharge was computed as 26,500 cfs. Recent data for the 25 years, 1914 to 1938, shows the average discharge of the Ohio River at Wheeling as 37,760 cfs some 30% greater than computed by Ellet.

In November 1850 Ellet was appointed to investigate the means of protecting the Mississippi Delta from inundation. His party made some measurements of slopes, cross sections, and velocities in the Lower Mississippi.

Ellet used surface floats at different points of a 500 ft reach. He made experiments with the purpose of finding the relationship between the surface and the mean velocity, for the computation of discharge. Using floats submerged 25 ft, 50 ft, and 75 ft he found that in the Mississippi River the maximum velocity is "in some point about midway between the surface and the bottom, therefore the mean velocity is about two per cent greater than the average surface velocity." He simply assumed that the measured surface velocity is equal to the mean velocity. Ellet derived his own binomial formula for the computation of velocity as a function of the maximum depth and the slope.²⁰

Ellet expressed an interesting opinion concerning hydrometry:²¹

"We are but little aided in determination of the facts attending the drainage of a country, and the discharge of its rivers, by the previous labor of philosophical writers. No systematic experiments on a large scale, with a view to the determination of the daily and annual discharge of great rivers, and the comparison of that discharge with the annual fall of rain for the climate, so as to obtain the amount consumed by vegetation and evaporation, over wide areas, have ever been instituted."

Presenting the data of the Ohio River runoff for six years, Ellet expressed²² his belief that such a work was unique, "never to have been made for any other river with equal care and accuracy, if, indeed, any authentic experiments of the kind have ever before been instituted at all."

THE MISSISSIPPI DELTA SURVEY, 1851 - 1860

A landmark in the study of rivers was reached in November 1851 when following an Act of Congress the Bureau of Topographical Engineers, War Department, appointed Humphreys to conduct a topographical and hydrographical survey of the Mississippi Delta. Forshey was invited to lead the hydrometrical party. The instruction listed kinds of works to be "commenced and prosecuted": the determination of a transverse section of the river with the utmost care and precision, the average velocity of the river currents, corresponding to each of the different stages, the duration of each stage, the amount of water

²⁰ Ch. Ellet, Jr., Report on the overflows of the delta of the Mississippi. Washington 1852, Congressional Documents No. 614, pp. 13-106.

²¹ Ch. Ellet, Jr., The Mississippi and Ohio rivers. Philadelphia 1853, 367 p.

²² Ellet was not entirely correct in this point. As far as is known, the first step in runoff computation was made by the Swiss professor Hans Conrad Escher for the Rhine River at Basle, 1809 to 1820. He applied the empirical formula by Eytelwein, checked it by one float measurement and assumed a constant slope, prepared a discharge table for every half foot and computed daily discharges (*Estimation de la masse d'eau fournie annuellement par le bassin du Rhin dans la partie suisse des Alpes*. Bibliothèque universelle des sciences, belles-lettres et arts, vol. 17, 1821, No. 4, Août, pp. 274-283. Genève). The French engineer André Baumgarten was the first to construct a discharge curve, in 1840. The famous Hungarian engineer Páló Vázarhelyi applied such a curve to the Danube in 1841. The Italian engineers Giuseppe Venturoli and San Bertolo computed the runoff of the Tiber River at Rome, 1822 to 1849, and published data in 1860. This data was reprinted by Nathaniel Beardmore in his "Manual of Hydrology", London 1862. Nevertheless, Ellet was the first man in America to study this problem, as well as to apply an equation of a discharge curve, and to compute the runoff.

conveyed annually through the river channel, the magnitude of the largest volume which could pass through the channel without overflowing the banks of the river.

The survey was launched "with great industry", but was interrupted by the sickness of the leader. The zeal of Humphreys induced him to remain so long and so late in the field that he became seriously ill. He returned to Washington and after recovering was sent to Europe to study waterways. The survey was resumed in 1857 when H. L. Abbot was appointed as the assistant to the Survey. Investigations continued and were finished in 1860 with the publication of an impressive report.²³ In its 610 pages and 20 plates the large volume contains a study of the history of river hydraulics, an experimental theory of water flow, methods of investigations and their results, and some suggestions for protection against floods.



FIG. 15.—A. A. HUMPHREYS



FIG. 16.—H. L. ABBOT

This great American treatise overshadowed all previous hydrographic works and became famous all over the world. Abstracts were translated into German, French, Italian and Russian. H. Grebenau included the theoretical part into his "Theorie der Bewegung des Wassers in Flüssen und Kalänen", Munich 1867. Extensive discussions were published in Europe by G. Hagen, A. Wiebe, G. Heidner, Grabenau, K. R. Bornemann, A. Treuding, G. Wex, C. K. Aird, W. R. Kutter, and F. Grashof. They were continued in the United States by D. Farrand Henry, S. W. Robinson, J. B. Eads, R. E. McMath, and also by both of the authors.

²³ A. A. Humphreys and H. L. Abbot, Report upon the physics and hydraulics of the Mississippi River. Professional Papers of the Corps of Topographical Engineers, No. 4. Philadelphia 1861, 610 p., 20 pl. Second edition: Washington 1876, 712 p., 25 pl.

Double floats were used exclusively for velocity measurements in the Mississippi River investigations. Velocity distribution along the verticals was investigated and a parabola with a horizontal axis below the surface was accepted as a law of velocity distribution. "The curves indicate the existence of a law, although the discrepancies are too great to permit the deduction of any algebraic expression for it." A one-point or mid-depth method was suggested for routine measurements. Correction to the mean velocity being taken from 0.963 to 0.986, depending on the magnitude of velocity.

Humphreys and Abbot derived a complicated formula for the average velocity as a function of the slope and the cross section. In 1869 it inspired two Swiss engineers, E. O. Ganguillet and Kutter, to combine American data with that from Europe and establish their renowned formula which was widely used in many countries for almost 80 years.²⁴

The measurements of the Mississippi brought out the fact that the river discharge is up to 20% greater at the same stage when the flood wave is rising, than it is during the falling stage. A loop was obtained instead of a curve. The same fact was found later in the Danube River in Hungary and in the Volga River in Russia. Humphreys and Abbot simply took an average curve for their runoff computations. The mean monthly water stage was converted directly into the average monthly discharge. The average runoff during the years 1851-1858 was computed to be 675,000 cfs. It corresponds to 7.37 in. of runoff depth from the total drainage area of 1,244,000 sq miles. Assuming the average rainfall of 30.4 in. over the entire area, the runoff coefficient was computed as 0.243.

This monumental work on the Mississippi River still retains its historical importance. Particularly valuable is Chapter III, State of Science of hydraulics as applied to rivers, which is "partly original and partly compiled from similar notices by Rennie, Lombardini, Storrow and others, and from various encyclopedias".



FIG. 17.—D. FARRAND HENRY

CURRENT METERS AND DANIEL FARRAND HENRY

During the 19th century discharge measurements in America were usually made by floats. In Europe current meters were preferred.

A current meter of a screw type was developed by a German engineer R. Woltman in 1790 and continuously improved by L. G. Treviranus in 1820,

²⁴ Further investigations by I. E. Houk in 1918, E. Beyerhaus in 1921, H. Lang in 1931, showed that the slopes, on which the Mississippi formula was based, had been determined erroneously, ten times too low. As a consequence the popular Swiss formula was discarded.

P. Boileau in 1845, A. Baumgarten in 1847, J. Amsler-Laffon in 1872, A. R. Harlacher in 1881. A current meter of this type, manufactured in London by an American mechanic Joseph Saxton, about 1836, was imported to the United States and tried on the Lower Mississippi in 1858, but did not perform satisfactorily in silt-laden water. Similarly, Allan Cunningham rejected the unsatisfactory current meter and carried on his famous investigations on Indian canals in 1874-1879 with double floats and floating rods.

The first American who constructed and successfully applied his current meter, was Henry. He also introduced several important improvements into the methods of measurements. Uncommonly talented since childhood Henry surpassed his contemporaries, but his ingenuity was not recognized. He is better known in Europe, although his name is often misspelled in the European literature, as Farrand Hay or D. H. Farrand. While engaged in outflow measurements of the Great Lakes into the St. Clair, Detroit, Niagara and St. Lawrence Rivers, Henry found floats unreliable in conditions of irregular flow. In 1867 he designed and made a "telegraphic current meter", outfitted with an electric circuit transmitting signals of its rotation. Henry tried runners of two different types. The first was a screw engaged in a cylindrical ring, the second had four hemispherical cups rotating on spokes around an axis transverse to the current, actually using the flier (rotor) from one of the T. R. Robinson's anemometers.²⁵ An electric contact was made once each revolution of the runner, and the signal was transmitted by insulated wire to a battery and automatic recorder, similar to the Morse telegraph. This innovation was extremely important, because it was no longer necessary to lift the current meter to the surface after every observation to read the counter. Electric transmission and recording allowed longer observations at every point for investigation of stream pulsation and provided perfect control during the work.²⁶

Instead of a supporting rod to keep the current meter fixed at selected points, Henry introduced a steel line, stretched out between a heavy weight lying on the bottom and the observer's stand. The water impact on the rod was considerably reduced in this manner.²⁷ The innovations introduced by Henry were very significant. He should be credited as one of the most eminent stream-measurement pioneers in the United States, although he has been virtually forgotten in his own country.

In his reports Henry emphatically condemned floats, which move faster than the water in which they are immersed. He recommended that all discharges measured by double floats be reduced by 10%.

Abbot, of the famous Mississippi River team, was obliged to defend the methods used by the Delta Survey. He recognized, however, the superiority of

²⁵ D. F. Henry, On the flow of water in canals and rivers. The Journal of the Franklin Institute, 62(1871), No. 3, 4, 5, 6; 63(1872), No. 1, 2 and 4. Philadelphia. Separate edition, with appendices: Flow of water in rivers and canals. Detroit 1873, 86 p., 11 pl.

²⁶ In Europe the electric transmission was adapted in 1872, when prof. Ch. M. Rühlmann suggested that the Swiss manufacturer J. Amsler-Laffon introduce such an innovation. Still earlier, about 1859, a French engineer Ch. Ritter inspired Salleron, a mechanic in Paris, to arrange a galvanoscope for recording signals sent by current meter with electric contact. This instrument was unsuccessfully tried in the Bosphorus Strait, in salt water. A. R. Harlacher, who first introduced electric signalization into European practice in 1871, recognized the precedence of Henry's invention.

²⁷ The idea of a flexible rod was tested in Europe several times, and is again becoming popular.

more modern instruments, and is to be credited with the introduction of another current meter with electric signalization. This current meter was used in the sea water of San Francisco Harbor in 1877-1878. Abbot's meter was of a screw type with a horizontal axis of rotation. The axle entered a closed gear-box, filled with oil, which protected the electric contacts against salt water. One half of the axle was made of agate, the other half of platinum. A spring touched the rotating axle and interrupted the current for half of each revolution.²⁸ A current meter of almost identical type was constructed by G. F. Deacon in Great Britain about 1881. It had changeable contacts and gear-box filled with liquid paraffin or mineral oil.

TH. G. ELLIS AND THE CONNECTICUT RIVER

Rapid progress in methods of hydrometry can be seen in the investigations of the Connecticut River in 1870-1873. The War Department commissioned

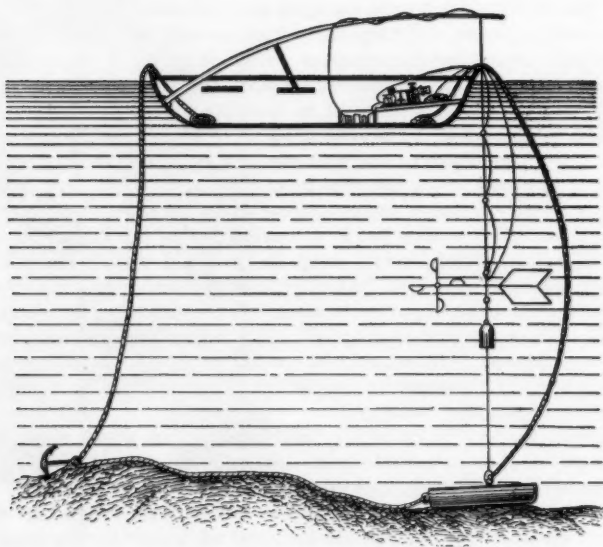


FIG. 18.—VELOCITY MEASUREMENT BY HENRY

²⁸ Current meter with a horizontal axis, but without electric contact, was used by J. J. Révy in another part of the Western Hemisphere, on the great rivers of South America, in 1870-1871. Julian John Révy was a Hungarian engineer in the service of the British government. Révy attached his meter to the end of a horizontal pipe, lowered on two cables by bifilar suspension. He applied an integration method of velocity measurement shifting the meter uniformly along the vertical. Révy published the results of his expedition in the book "Hydraulics of Great Rivers - the Paraná, the Uruguay and the La Plata Estuary", London 1874. This book became famous, like the Humphreys-Abbot Mississippi Report. It was abstracted in all technical periodicals of that time. In 1874 the Van Nostrand's Eclectic Engineering Magazine in New York reprinted four reviews.

Th. G. Ellis to this work. Humphreys was at that time Chief of Engineers, U. S. Army.

Ellis selected the site for measurements near Thompsonville, Conn., above the Enfield Rapids, beyond the influence of tidal fluctuations. He had to use double floats, but preferred current meters. One meter of the Baumgarten type, manufactured by Secretan in Paris, was imported from Europe and improved by Clemens Herschel in Boston. The second meter was constructed by Ellis himself, following the advice of Henry. Four conical cups were installed instead of hemispheres. The electric contacts were similar to that of Henry's meter. This type became known as the Ellis meter.

Ellis investigated velocity distribution along many verticals both with double floats and with a current meter. Floats on 84 verticals showed on the average 11.4% greater velocities. Velocity distribution diagrams, 1266 in number, were planimeted and average velocities computed. A correction factor of 0.933



FIG. 19.—T. G. ELLIS

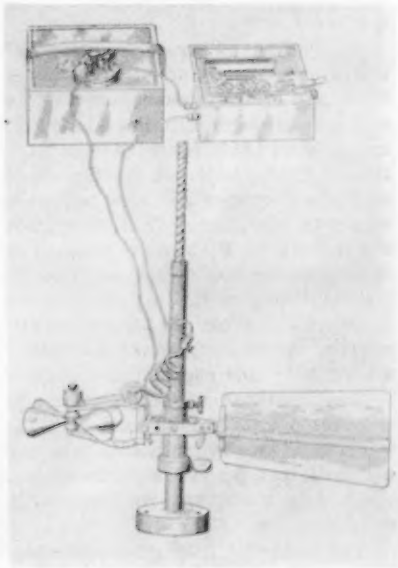


FIG. 20.—ELLIS METER

was determined for the mid-depth velocity: this method was originated on the Mississippi River. Ellis also tried the method of integration along the verticals, in a similar way to that which Revy did at the same time. A few years later this method was introduced in Europe by Harlacher. Ellis confirmed the parabolic law of velocity distribution, with a maximum below the surface. He used current meters, therefore his results were more accurate and convincing than those previously obtained on the Mississippi with floats. A discharge curve was constructed for the Connecticut River, with two branches—for rising and falling stages. Daily discharges were computed for the years 1871 to 1877. The 7-yr average discharge was found to be 20,100 cfs. The present value is 16,150 cfs, or 25% less.

UNITED STATES COAST SURVEY

Hydrometric elements in hydrographic work at sea include stage (tide) observations, depths and current measurements, and sampling of water and bottom specimens. This area in the United States is represented by the Coast Survey, established in 1807, actually revived in 1832, and since 1878 known as the U. S. Coast and Geodetic Survey.

The first Superintendent of the Survey was Ferdinand R. Hassler, a renowned Swiss geodesist, who developed geodetic and hydrographic work in extremely difficult circumstances. He started studies of tides and currents. His successor, A. D. Bache, the first President of the National Academy of Sciences, was a promoter of many technical improvements. He proposed a method of graphical presentation of currents on nautical charts. Series of lines were drawn at distances inversely proportional to the velocities, similarly to the stream lines in the flow nets. In 1847 J. N. Maffitt applied this method in the Boston harbor.

Many devices for depth sounding were invented and improved. M. F. Maury introduced wire instead of hemp sounding line. It is known that Governor Winthrop of the Massachusetts Bay Colony made efforts to obtain soundings and to take water samples along the North Atlantic coast of the United States as early as 1663. He tried to use a sounding device without a line, invented by Robert Hooke in Great Britain. In 1854 J. M. Brooke, cadet of U. S. Navy, invented a deep-sea sounding bob with a dumping weight.²⁹ This ingenious device was modified by O. H. Berryman and B. F. Sands in 1857. Sands also used the method by E. Massey for sea-depth measurement: a screw-type meter measures the way of the sounding weight, the reading of the number of revolutions replaces the measurement of the length of the wire. In 1859 W. F. Trowbridge improved this method: he adopted a mechanism of the Saxton current meter for greater accuracy. Two meters were attached to a frame of a plummet: one registered movement down, another up.

An apparatus with an air bag and manometer was designed by E. B. Hunt and tested by W. G. Temple in 1857, long before the pneumatic principle was introduced into hydrometric practice by W. Seibt in Germany, in 1898. Hunt also anticipated a hydraulic transmission of the pressure difference, which was adapted by the Hungarian engineer S. Hajós to his "hydrostatic profilograph" in 1904.

These sounding devices were displaced by the sounding machine invented by William Thomson (Lord Kelvin) in 1872. C. D. Sigsbee improved and introduced this machine into the practice of the Coast Survey.

Instruments for observation and transmission of water-level fluctuations were developed for various conditions. In 1857 S. D. Trenchard designed a display gage: a cotton belt, connected to a float, passed over a roller and stretched by a counterweight. A number on the belt, readable from a distance, showed the actual water stage through a window. In 1857 H. Mitchell made a swinging gage for the open sea: 1 60-ft long pine spar was fixed at the bottom by a hinge, the upper part was floating in an inclined position. A pendulum at the top helped to read the angle of inclination of the spar for computation of the depth. The length of the submerged part could be read exactly in a glass tube attached

²⁹ Russians claim that a similar device was invented by the first Emperor of Russia, Peter the Great, about 1710. Actually, G. Almé, a French oceanographer, devised such a sounding instrument in 1843 and applied it at the coasts of Algeria.

to the upper part of the spar; a colored glass bubble was an index for reading.

A self-registering gage, the first in America, was constructed in 1845 by Saxton, a famous mechanic of precision instruments, and introduced into practice by Hunt in 1853. The instrument had a wire transmission from a float to a pencil, and a pendulum clock for the shifting of a paper sheet. This gage was in operation at the mouth of Southwest Pass of the Mississippi River from May 1859 to June 1860, during the Delta Survey. A more enduring mareograph was built by R. S. Avery, who also wrote a detailed manual on "Methods of registering tidal observations" in 1876.

Floats were generally used for the observation of currents. In 1877 H. L. Marindin described their use in the investigations of the Mississippi River outlets: a simple can-float for surface velocity and a double float for measurement at depths. A loaded barrel was connected by a piano wire to an ellipsoidal



FIG. 21.—J. B. FRANCIS

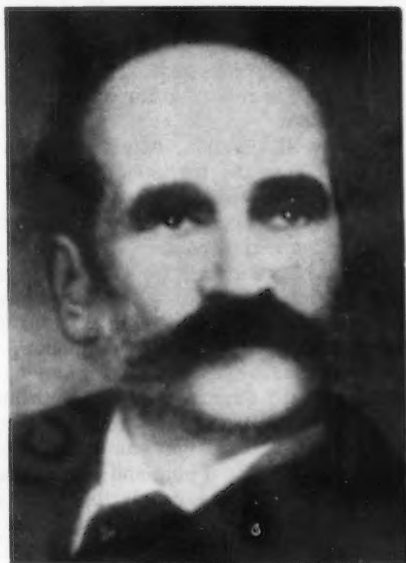


FIG. 22.—A. FTELEY

surface float. No corrections were requested for these measurements. Marindin expressed his opinion that the only way of obtaining reliable data would be the introduction of an electric current meter. In 1891 E. E. Haskell tested his direction-current meter, which he constructed with E. S. Ritchie, in the Florida Straits.

In 1849 G. M. Bache invented a self-closing deep-water sampler. In 1860 Mitchell made a device for collecting specimens from the bottom in alluvial harbors. Sigsbee devised a clam-bucket type bottom sampler.

"CLASSIC PERIOD" OF NEW ENGLAND

C. W. Sherman honored the 16 following "great hydraulic engineers of New England's classic period": L. Baldwin, Storrow, J. B. Francis, U. A. Boyden,

E. S. Chesbrough, J. H. Shedd, J. F. Davis, H. F. Mills, T. G. Ellis, A. Fteley, J. T. Fanning, Herschel, D. FitzGerald, E. B. Weston, F. P. Stearns, and J. R. Freeman.³⁰ Boyden is known for his many improvements to water wheels and sometimes is credited with the invention of a reaction turbine. He introduced the hook gage, an excellent device widely used in hydraulic laboratories.

The most renowned among Bostonians is Francis, the inventor of the Francis turbine, which he patented in 1847. In 1915, at the occasion of the 100th anniversary of his birth, K. Keller, a German engineer in Munich, wrote³¹ exaltedly: "As long as water shall find its way to the valleys, there to be used in turbines, the name of James Bicheno Francis will stand as that of a great hydraulic engineer."

Francis was chief engineer of the Merrimac Company for 48 yr (1837-1885). In order to correctly distribute available water among various mills, Francis made many experiments on the flow over weirs and in regular channels, and tests of hydraulic motors. He discovered that many, if not all, of the mills were using more water than they were entitled to under their water-power rights. Daily records of the Merrimac River flow were kept by Francis at Lowell, Mass., from 1848. They are, however, not complete, because the water discharge was recorded for only a 10-hr period each day, when the mills were in operation.

In Lowell, Francis established the first hydraulic laboratory for turbine testing. Then the turbines could be used as water meters. He calibrated a 14-ft long rectangular weir with side contractions and derived a convenient formula, occasionally used even now. This pioneer investigation was published in his classical report.³² Francis used rod floats in his measurements in channels. His formula for correction to the average velocity, published in the third edition of his work, was well known.

Two other engineers from Boston, Fteley and Stearns, made many hydro-metric measurements for the Sudbury aqueduct of Boston between 1875-1880. A rectangular weir was used for discharge measurements. Efforts were made to improve the Francis formula, with only limited success, however. A current meter of the Ellis type was found unsuitable in those conditions and an instrument of original design was constructed. It had a fanlike runner of 6 to 10 blades fixed inside a ring and rotating around a short horizontal axle. Electrical indication of the revolutions was applied. The Fteley-Stearns current meter was sensitive, but delicate, more suitable for laboratory conditions, and did not find broader acclaim. During calibration of the meter for the first time its action was investigated at oblique angles. In 1876 a recording gage, made by Fteley, was put into operation. In 1881 the results of 5-yr of observations were published.³³ They were praised as the most accurate and systematic in the United States up to that time.

³⁰ C. W. Sherman, Great hydraulic engineers of New England's classic period. *Engineering News-Record*, 107(1931), No. 13, Sept. 24, pp. 475-479.

³¹ Tribute to James B. Francis (Editorial). *Engineering Record*, 72(1915), No. 24, Dec. 11, p. 711.

³² J. B. Francis, *Lowell hydraulic experiments*. Boston 1855, 156 p. 15 pl.; 2 ed. 1868, 286 p. 23 pl.; 3 ed. New York, 1871, 4 ed. 1883, 5 ed. 1909. A part of this work was translated by K. R. Bornemann and published in the German magazine "*Der Civilingenieur*", 2(1856), No. 6, pp. 163-186, in Freiberg, Saxonia.

³³ A. Fteley, The flow of the Sudbury River, Massachusetts, for the year 1875 to 1879. *Transactions, ASCE*, 10(1881), pp. 225-250.

In 1883-1885 Stearns investigated water flow in the Boston drainage conduit. A current meter mounted at the end of a rod was introduced into the conduit through a manhole. A special device was designed to keep the meter at any desired point of the conduit section.

Another Bostonian, Herschel, gained world-wide fame by the invention of the venturi meter in 1887. As a hydraulic engineer for the Holyoke Water Power Co. in Massachusetts, Herschel was required to measure the water supplied to a hundred textile mills. About 1880 he arranged the famous Holyoke testing flume, where he tested turbines, which were later used as water meters. Part of the water was drawn by pipes to some twenty-five large paper mills and used for washing. Looking for the possibility of measuring the water flow in pipes, Herschel incorporated a throat into a pipe. Simple manometer readings gave the difference in pressure in the pipe and in the throat, and the

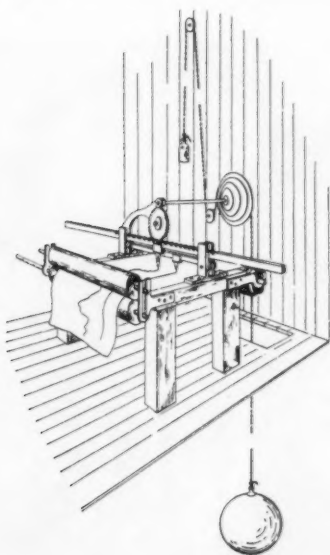


FIG. 23.—RECORDING GAGE USED BY FTELEY IN SUDBURY RIVER, 1876

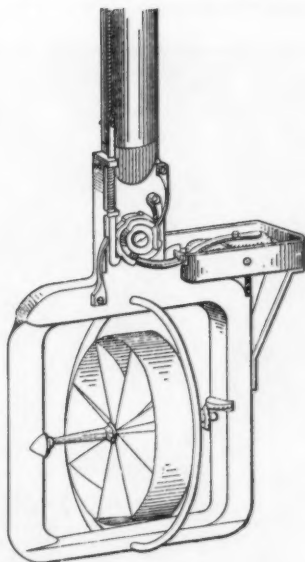


FIG. 24.—FTELEY-STEARN'S CURRENT METER

discharge was obtained therefrom. Herschel checked this device in a 1-ft pipe and in a 9-ft pipe. He called it the venturi tube in honor of Giovanni Battista Venturi, Italian scientist (1746-1822), who first demonstrated the phenomenon of pressure decrease in a throat in 1797. A diffuser was added to regain the loss of head.³⁴

The venturi meter proved to be a convenient and an accurate device for water measurement and recording. The two oldest venturi meters were installed into the water main of Newark, N. J., in 1892. They were in operation

³⁴ C. Herschel, The Venturi water meter. Transactions, ASCE, 17(1887), pp. 228-258; discussions: 18(1888), pp. 133-141.

without any failures for 60 yr, up to 1952.³⁵ In 1912 three venturi meters, the largest at that time, were built in the Catskill aqueduct of the New York City water supply. A throat of 7.75 ft was located in the concrete tube 17.5 ft in diameter, 111 ft long. In 1929, when Herschel was 87 yr old, he wrote³⁶ his dramatic "Farewell word." At that time more than 40,000 venturi meters were in operation. The renowned water engineer J. J. R. Croess, at that time President, ASCE, said in 1900, that the invention of the venturi meter was sufficient to make the last quarter-century noteworthy in the annals of scientific progress.

Furthermore, in 1920, Herschel proposed an improved type of overflow weir for river gaging: a submerged weir with the upstream face sloping 2 to 1 and a 2-ft wide crest. A perforated horizontal tube was to be laid along the crest for head measurement and a device provided for automatic correction of the velocity of approach.

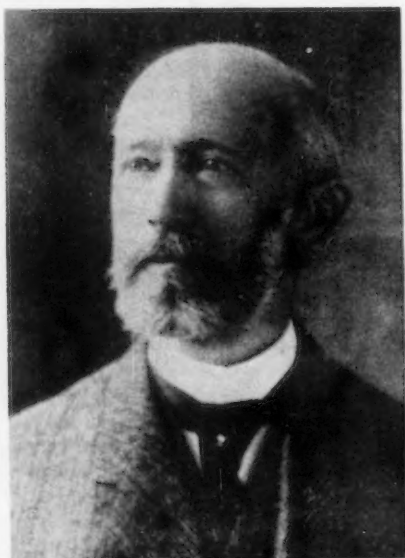


FIG. 25.—F. P. STEARNS



FIG. 26.—C. HERSCHEL

Hydrometry is indebted to Herschel for one more achievement. In 1897 he visited Italy and found in the monastery of Montecassino an old Latin manuscript "De aquis urbis Romae libri II", written by Sextus Julius Frontinus, the water commissioner in Rome, about 97 A.D. Herschel procured a photographic copy of this interesting document, translated it into English and published it with facsimile and comments.³⁷ Herschel disclosed the meaning of the *quinaria*,

³⁵ L. M. Leedom, Two old veterans retired by Newark. *Water & Sewage Works*, 99 (1952), No. 2, Feb., pp. 63-67. Chicago.

³⁶ C. Herschel, A farewell word on the Venturi meter. *Engineering News-Record* 102(1929), No. 16, Apr. 18, pp. 636-637.

³⁷ C. Herschel, The two books on the water supply of the city of Rome of S. J. Frontinus. Boston 1899.

a unit of water quantity used in old Rome. Actually it was the diameter ($5/4$ of a digit, $1/16$ of a Roman foot) of the nozzle, carix, which took water from the aqueduct or reservoir, regardless of water pressure and velocity of the outflow. The gravestone of Herschel in the cemetery at Watertown, Mass., bears an inscription, taken from Frontinus: "Memoria nostri durabit, si vita meruimus" (Remembrance will endure if the life shall have merited it).

MISSISSIPPI RIVER INVESTIGATIONS AFTER 1877

Continuous expansion of the population and development of waterways soon attracted renewed attention of the government to the great rivers. In 1877 the Corps of Engineers, U. S. Army, reestablished regular hydrometric work on



FIG. 27.—OHIO RIVER AT PADUCAH, 1882

Mississippi River. A number of permanent gaging stations were opened and discharge measurements organized in several sections. The few reports were published only in Congressional Documents.

Rod floats were still used, at least up to 1900. E. Burr and W. S. Mitchell made their measurements by rod floats at St. Louis, Mo., in 1900. At that time it became apparent that float measurements, with 11 to 16 men engaged, were more expensive than those with current meters. Soon float measurements were entirely discontinued.

Extensive research work was done by the Army engineers with current meters at several sites. The purpose was to establish the form of velocity distribution along the verticals and to check the various approximate methods. Velocities were measured at many points, curves were drawn, and average curves were derived. During 1882, for example, over 1000 verticals were

examined, as follows: Ohio River at Paducah, Ky., 211; Mississippi River at Columbus, Ky., 101; at Helena, Ark., 90; at Red River Landing, La., 32; at Hays Landing, Miss., 620 verticals. It is regrettable that these valuable data were never used for the advancement of science of river hydraulics.

An extremely interesting experiment was undertaken by Army engineers in the Mississippi River near Burlington, Ia., in 1879. Velocity variations were investigated simultaneously by six current meters along a vertical. The meters were of the cup type, designed by Ellis, made by Buff and Berger in Boston. A weight of 150 lb was lowered from a securely anchored boat. A wire line was stretched between the bottom weight and the boat, with six meters mounted and equally spaced. Current meter revolutions were registered on an especially constructed electric chronograph, which had eight pens; two were used for time signal recordings. The meters were allowed to run for 11 min to 32 min during each set of observations. Records for every minute were taken



FIG. 28.—W. G. PRICE

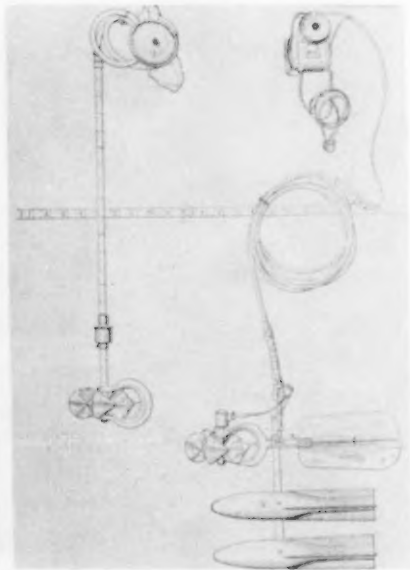


FIG. 29.—PRICE METERS

from the registration tape. Vertical velocity curves determined from long observations showed an extremely regular character, and a maximum velocity at the surface, contrary to all previous investigations. The correction factor for the mid-depth method was determined as 0.958.

The observations at Burlington were unquestionably the first investigations of pulsation to such an extent. The engineers were aware that they observed velocity components from all directions, because their meters had free orientation. The cup meter is not suitable for measurement of a required projection of velocity. However, the general character of pulsations was correctly ascertained. The same laws were later found in Europe by Harlacher and in Siberia by Russian engineers. The most surprising fact was that similar fluctuations occur at different depths at the same time. Therefore the influence of pulsation

cannot be eliminated by short observations at a great number of points, as it is generally believed.

The work at Burlington was performed by G. A. Marr, assistant engineer. The report was published by A. Mackenzie in what is now an extremely rare booklet. Abbot wrote in introduction to this report:³⁸ "This report covers the most exact set of measurements which has come to my knowledge . . . The plan of simultaneous measurement at different depths by electrical meters is a decided advance over any of the older methods, and is novel and of much scientific interest."

However, the current meters used at that time still had many weaknesses. W. G. Price, employed by the Mississippi River Commission was dissatisfied with the Ellis meter and constructed his own in 1882. It was a meter of a cup type with five conical cups and simple bearings protected from water and silt. This Price meter was patented in 1885, became very popular and is still in use in an improved form.³⁹ Price described equipment of hydrometric work and methods of observation in his article in 1898.⁴⁰

Daily stage observations along the Mississippi River have been published since 1871.

GREAT LAKES INVESTIGATIONS

Another branch of the Corps of Engineers, the U. S. Lake Survey, created in 1841, with its center in Detroit, Mich., has carried out some very important hydrometric work on rivers connecting the Great Lakes. Regrettably little has been published about these interesting and valuable investigations. Records of lake levels are intermittent from 1815 and continuous since 1859. Thirty-two gages are in operation between Duluth and the St. Lawrence River; most of them are automatic. The first hydrometric work, already mentioned, was done by Henry in 1867-1869. More significant measurements were accomplished by J. C. Quintus in 1893, C. B. Stewart and Haskell in 1897-1898, F. C. Shenehon in 1898-1902, L. C. Sabin in 1899, Murray Blanchard, F. ASCE, in 1900-1902, T. Russel in 1902-1903.

The conditions for hydrometric measurements in the lake outlets were different from the Mississippi River. The bottom is stable, the water is clear, the currents are swift. Variations of discharge during a year are minor. However, the use of stages for daily discharge computations is extremely difficult, as the influence of wind and variable backwater effect complicate normal relationships.

³⁸ A Mackenzie, Report on current meter observations in the Mississippi River, near Burlington, Iowa, during the month of October, 1879. Washington 1884, 38 p., 42 plates.

³⁹ Actually, the first current meter with a vertical axis of rotation was invented in Italy. In 1806 Francesco Focacci constructed a "molinello" with a vertical axle, located in a long tube, with which it was immersed into the stream. The rotations of the wheel were directly transferred to a dial above the water surface. In 1823 the engineer Antonio Sempiterni Tolotti applied a similar runner with eight blades which rotate in a protective case. Generally, however, the cup type meter had never appealed to European engineers.

⁴⁰ W. G. Price, Gauging of streams. Journal of the Western Society of Engineers, 3(1898), No. 3, May and June, pp. 1025-1040. Chicago.

The Price meters were manufactured by a firm W. & L. E. Gurley in Troy, New York. Therefore in Europe the Price meter is often called the Gurley Meter.

The Lake Survey used current meters with a horizontal axis. In 1886 Haskell, later Dean of Engineering at Cornell University, developed a current meter of a screw type with three blades and with an electric transmission. The bearings and electric contacts were not protected against water. In 1890 another type was designed by Haskell with the cooperation of Ritchie, of compass fame. It was the Ritchie-Haskell direction current meter. Signals indicating deflection of a magnetic needle sealed inside the instrument were transmitted electrically to a meter-direction indicator, when suspended free under water surface. The Ritchie-Haskell meter was used in sea-currents measurements. In 1913 it was applied by a Russian engineer A. E. Korovin in the Volga River for the investigation of directions of the currents.

Many innovations were introduced into practice in the Lake Survey by Shenehon, who was later Dean of Engineering at the University of Minnesota. He invented the method of index point. The measured discharge was correlated to the velocity in a certain significant point. Further daily-discharge computations were based on the control velocity observations in the index point only. In complex profiles, as between bridge piers, several index points were necessary in different parts. Shenehon derived a correction table for cable deviation at different depths and angles of deflection. This table, although not particularly correct, was adopted by the U. S. Geological Survey. In 1902 Shenehon invented the wire sweep, a wire suspended under floats at a given depth and towed by a vessel at each end. It serves to reveal underwater obstacles and is now in general use everywhere.



FIG. 30.—C. B. STEWART

The hydrometric work done by Sabin⁴¹ in the St. Clair River deserves particular attention. Sabin applied eleven current meters simultaneously. The meters were mounted on a cable straightened between a 200 lb sinker at the bottom and the boat. The meters were of the Haskell type, two of them with direction indicators. The meters were equally spaced at every tenth of the water depth. Their revolutions were recorded by electric counters, operated by a common switch. In order to eliminate the influence of pulsation, several observations were continued for 100 sec at every point, and one observation was made for 600 sec. Sabin obtained perfect curves of velocity distribution for twenty-one verticals in one cross section. Velocity at the surface was a little distorted by the effect of catamaran hulls. The effect of wind was very slight and did not extend deeper than to two-tenths of the depth. The mean velocity along the vertical was found to be just below the six-tenth depth. Sabin adjusted an empirical equation of the ellipse, and hence computed a series of relationships between the velocities at

⁴¹ One shiplock in the St. Mary's Falls Canal (Soo Canal between the lakes Superior and Huron) is called the Sabin Lock in honor of the constructor of this canal.

different depths, with a high degree of accuracy.⁴² Sabin also tested the integration method by shifting a boat with a suspended current meter across the river.⁴³ Sabin drew curves of equal velocities in the cross section, the isochs, using derived average data. He did it for presentation only, not for determination of the discharge by the Culmann method.



FIG. 31.—E. E. HASKELL

The pulsation was investigated by simultaneous observation of several current meters located at various points across and along the river. Revolutions were registered during a period of 10 min, readings were taken every 15 sec. Fluctuations showed a variety of waves, not identical for the whole section and not synchronous at different points. No definite conclusions could be drawn.

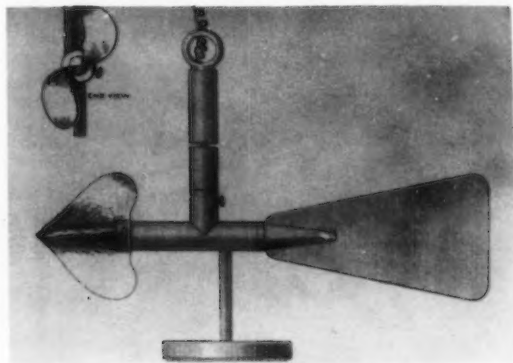


FIG. 32.—SMALL HASKELL METER

Blanchard performed similar work with the same set of eleven meters in the Detroit River in 1900. He investigated twenty verticals.

⁴² The elliptic form of the velocity curve was applied in Europe by A. von Gerstner in 1819 and by M. Raucourt in 1832. It was tried again by A. Wellner in 1933 and by W. Kresser in 1954.

⁴³ The horizontal integration was introduced in Europe by L. G. Treviranus in 1848. It was successful in artificial channels with horizontal bottom only.

These simultaneous observations with so many current meters were the first in the history of hydrometry. There had been some unconfirmed rumors that about 1945 investigations of tide currents were made for the Quoddy Tidal Development with twelve current meters mounted on a suspension cable in a 200-ft deep channel. The range of the tide there is 28 ft. Multiple meters are frequently used for testing hydroelectric power plants.



FIG. 33.—F. C. SHENEHON

Blanchard developed a method for the discharge determination in conditions of variable slope. A series of discharge curves is constructed for different values of the fall between two gaging stations, assuming that this fall is a definite function of the average stage at both stations. A deep river with stable

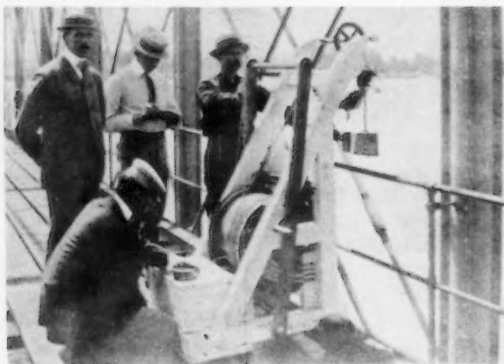


FIG. 34.—NIAGARA RIVER, INTERNATIONAL BRIDGE, 1899

channel and reasonable distance between gaging stations are presumed for a success of this method.⁴⁴

⁴⁴ M. Blanchard, A discharge diagram for uniform flow in open channels. Transactions, ASCE, 96(1932), Paper No. 1807, pp. 865-886. New York.

PROGRESS OF HYDROMETRY IN THE WEST

River investigations in the West, as previously mentioned, were a great adventure. The first regular hydrometric observations were established in California in 1878 and in Colorado in 1881.



FIG. 35.—L. C. SABIN

W. H. Hall, California State engineer, got an appropriation of \$100,000 for the investigation of the Sacramento and San Joaquin rivers for a period of two years. His assistant, C. E. Grunsky, conducted these measurements⁴⁵ in 1878-79. Grunsky investigated floats, disclosed many sources of errors, and gave preference to current meters. The meters were of the type as designed by

FIG. 36.—CATAMARAN OF THE U. S. LAKE SURVEY,
1899

Abbot and applied in California during investigations in San Francisco Bay by the Army Engineers in 1877-78, although Grunsky called them of the Henry type. The meters were used on cables, bifilarly suspended from a catamaran

⁴⁵ Report by C. E. Grunsky is known in Europe, because it was submitted to the Stuttgart Technical University for a degree and published in German language: Hydrometrische Messungs-Verfahren in den Vereinigten Staaten Amerikas. Zeitschrift für Gewässerkunde, 10(1911), No. 3, pp. 1-50. Dresden.

of two boats. A third boat was anchored in front of the catamaran and was used to stretch a stay-line in order to keep the frame with the meter in a perpendicular position. Observations were taken on ten verticals, velocities were measured at ten points on each vertical and continued for three minutes at every point. Smooth vertical curves were obtained.

Grunsky checked approximate formulas for velocities along the verticals and derived a formula for a ratio between the average and surface velocity on a vertical. He advocated, with little success, however, the graphical determination of the discharge measurements, analogical to the Harlacher method, generally used in Europe since 1881. Grunsky proposed a method of daily run-off computations for the rivers with an unusually variable bed, by tracing the curves of equal discharge on the diagram of stage fluctuations. This method was applied by W. B. Clapp to the Colorado River at Yuma, Arizona, in 1903, and by A. I. Tchórzewski on the Amu-Daria in Turkestan, in 1913. Apparently,



FIG. 37.—MURRAY BLANCHARD



FIG. 38.—C. E. GRUNSKY

Grunsky was the first hydrologist who applied the flood-routing procedure for the forecast of discharge variations downstream.

Hall introduced the method of plotting mean-velocity and area curves besides the discharge curve. Sometimes the separate analysis can disclose the source of discrepancies in the case of shifting channels. First applied in 1878, this method was forgotten until 1904, when G. F. Harley and F. W. Hanna rediscovered it.

Colorado hydrometric stations were taken as the pattern for the organization of the net of the U. S. Geological Survey stations. In 1881 E. S. Nettleton, the State engineer in Colorado, established the first gaging station on the Cache la Poudre and the Big Thompson rivers and measured the flow for several months.

A current meter was constructed of a cup type, known sometimes as the Colorado meter, and sometimes as the Lallie or Bailey meter. Recording gages and cableways were installed in Colorado about 1890. In 1889 R. Robertson built the first cableway on the Arkansas River near Canyon City. It was equipped with pulleys for lowering the observer's seat to the water surface. The first car placed at a fixed distance below the cable was installed by

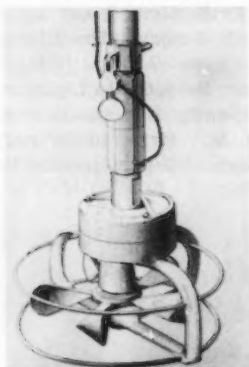


FIG. 39.—COLORADO
(BAILEY) METER

W. A. Farish on the Salt River in 1890. At the same time W. P. Trowbridge in accordance with Hall's ideas applied a traveler, an unmanned cable-car, for operation of the meter from the bank. The aim was to protect observers from endangering their lives, particularly during the floods. Such an arrangement was used experimentally on the Tuolumne River at Modesto, Calif., and tried in several other locations. Blanchard used a traveler in the Chicago Main



FIG. 40.—THE FIRST CABLE CAR, ARKANSAS RIVER AT CANYON, COLO.

Channel in 1915. C. H. Pierce still made some use of this arrangement in Connecticut and Vermont in 1916-19.

UNITED STATES GEOLOGICAL SURVEY

An Act of Congress opened a new chapter in the history of hydrometry in 1879. By this act the Geological Survey was created for the purpose of investigating the natural resources of the United States. Its founder and one of its directors was Powell, famous explorer of the canyons of the Colorado River. In 1889 a young engineer F. H. Newell was appointed to organize the Water Resources Branch. He well deserves the title of the "Father of systematic stream gaging".

Fourteen young men were selected to learn and to improve the methods of stream gaging. A training camp was established in December, 1888, on the Rio Grande at Embudo, N. M. Here, under rough field conditions, the first hydrographers of the Geological Survey passed their training. Newell, the 26-



FIG. 41.—F. H. NEWELL



FIG. 42.—E. C. MURPHY

yr-old leader, felt that he must wear whiskers in order to increase his dignity:⁴⁶

"From that camp, like the early Apostles, they went forth to spread the gospel of hope for the vast area of arid America, beginning the collection of data of stream flow which has since made possible the construction of the great projects for the irrigation of the Great American Desert."

The first steps of new hydrographers were hampered by the lack of funds and by political instability. The Water Resources Branch gained recognition and support, when irrigation works had spread spontaneously in the West.

⁴⁶ F. J. Seery in *The Cornell Civil Engineer*, 21(1913), No. 7, p. 412. Ithaca, New York.

These works finally absorbed Newell, who became chief engineer of the U. S. Bureau of Reclamation in 1902.

Thousands of gaging stations were installed in steps across the country. Methods and equipment were developed, skillful engineers trained. Data of observations and reports were published in a series of Water-Supply Papers, modest booklets of enormous significance. The first manuals on hydrometric techniques appeared among them: Methods of stream measurements, 1901, by Newell; Hydrographic manual of the U. S. Geological Survey, 1901, by E. C. Murphy, J. C. Hoyt and G. B. Hollister; Accuracy of stream measurements, 1902 and 1904, by Murphy. Murphy was the first scientist among these men of practice. His study was the first scientific evaluation of the contemporary hydrometric techniques.

N. C. Grover, a distinguished hydrologist, was the chief engineer of the Water Resources Branch from 1904 to 1939. Together with Hoyt he published

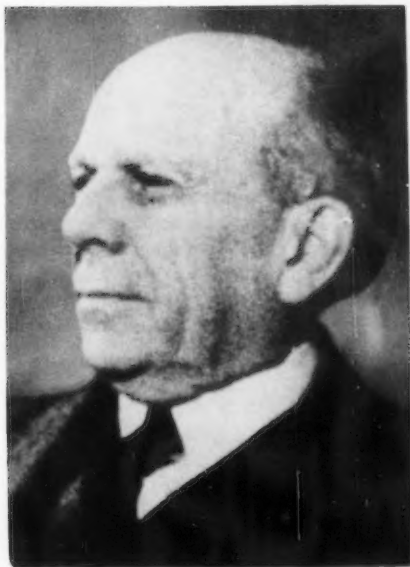


FIG. 43.—N. C. GROVER



FIG. 44.—J. C. HOYT

"River Discharge", an excellent textbook on American methods of hydrometry, widely used here and abroad. Four editions were issued from 1907 to 1916. Great credit belongs to R. Follansbee and Grover for an extremely valuable monograph "A history of the Water Resources Branch of the United States Geological Survey to June 30, 1919". This book was issued unofficially in 1938. The second volume still waits publication. It features a very fascinating story of the heroic efforts of the pioneers in the establishment and progress of this great organization.

The following excerpt from another report⁴⁷ vividly describes the typical living conditions of pioneer-hydrographers:

"Up to the advent of automobile roads as we know them today, the field men traveled by train. The engineer left headquarters for a routine circuit of the gaging stations assigned to him, with a current meter outfit in one hand and a

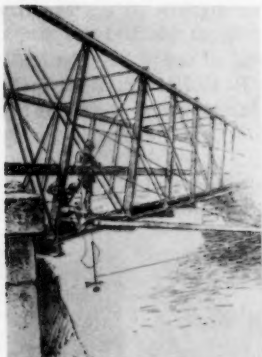


FIG. 45.—MEASUREMENT
FROM A BRIDGE

suitcase in the other; and sometimes he had also a small engineer's level, tripod, and rod, besides rubber wading pants. His luggage weighed 50 to 150 pounds . . . Then trips lasted from two to ten weeks, depending upon the number of gaging stations in the circuit. He was going night and day trying to get the necessary data between train schedules, getting off and on trains at all hours.



FIG. 46.—EARLY CABLE CAR, SHENANDOAH RIVER,
NEAR MILLVILLE, W. VA.

If the gaging station was not near the railroad, he hired a rig and drove to it. If the gaging station was more than about five miles he probably had to spend the night at the gage reader's house".

Current meters were used almost exclusively in the practice of the Water Resources Branch. In 1897 E. G. Paul of this Branch improved the Price

⁴⁷ J. H. Morgan, Development of stream gaging in Illinois. The Illinois Engineer, 12(1936), No. 4, Apr., pp. 49-55.

type meter. This improved meter, called the "Small Price", has been the standard of the Branch since that time.

Many methods developed by the hydrographers of the Geological Survey are known and have been adopted in different countries all over the world. Universally known is the Stout method, introduced in 1898 by O. V. P. Stout for correction of observed stages to the discharge curve when the river bed is shifting. Less popular was the method proposed in 1907 by R. H. Bolster for adjustment of the discharge curve to unstable bottom. Well known is the method of construction and extrapolation of the discharge curve, originated by J. C. Stevens, F. ASCE, in 1907, as also the method by M. R. Hall of 1908 for variable slope corrections.

A standard method of velocity measurement was adopted. Two points are taken at eight tenths and at two tenths depth on every vertical in a cross section of the river.⁴⁸ H. K. Barrows of the Geological Survey carefully examined



FIG. 47.—O. V. P. STOUT



FIG. 48.—J. C. STEVENS

this "two-point method" in 1906. Since that time the method has been very popular, and is known abroad as an American method.

Barrows and R. E. Horton in 1907, and W. G. Hoyt in 1913, investigated the river flow under ice cover and methods for computation of the winter runoff.

The work of the Water Resources Branch was carried on in accordance with local conditions. In many locations cableways were installed for one-man operation. In 1913 C. C. Covert, G. J. Lyon and Pierce prepared "Planes and specifications for current-meter gaging stations". Recording gages have been in

⁴⁸ This method was first suggested by Major Allan Cunningham in India in 1883 and checked there on 565 sets of measurements.

use since 1903. Since 1913 automobiles have been widely used. In 1903 A. H. Horton introduced heavy weights for use with a suspended current meter. Reels and cranes were developed by C. H. Au for manipulation with heavy weights from bridges and cablecars.

A special division of Ground Water was under the leadership of a renowned hydrogeologist O. E. Meinzer. M. O. Leighton was known in the field of hydrogeology by his methods of field tests and C. S. Slichter by the electrical methods of ground-water investigations.

WEIRS AND DAMS

Weirs are convenient devices for quick and accurate water measurement. A bulkhead is put across a channel, a sharp-edged notch is adjusted, the head



FIG. 49.—H. K. BARROWS



FIG. 50.—R. E. HORTON

over the crest is read, and the water discharge is easily computed. An appropriate discharge coefficient has to be selected or established by tests in a hydraulic laboratory. Spillways of existing dams can be successfully used for discharge determination, if a correct discharge coefficient can be assumed taking into account all local peculiarities, or determined by calibration.

G. F. Rafter used dams during his investigations in the State of New York in 1898. He based his work on extensive tests performed by H. Bazin in France, 1888-98, and also used the results of the tests at the Cornell University, collected by G. S. Williams and R. E. Horton. He published a study on methods of

computing the flow over dams.⁴⁹ R. E. Horton compiled all available empirical data on weirs for use by the hydrographers of the U. S. Geological Survey.⁵⁰ He also collected results of turbine tests, necessary to compute the flow through turbines in connection with the gaging stations at the dams.⁵¹

MEASUREMENTS IN PIPES

Investigations of the flow in pipes were made by many Americans in search of empirical formulas and roughness coefficients. Mills was the first to use a pitometer for studies of water flow in a 12 inch pipe at Lawrence, Mass., in 1877. He located 16 to 18 nozzles in one section. Each was connected to a manometer and all could be read at once. This work was continued by Freeman in 1878-83. Careful experiments, performed by Freeman in Nashua, N. H., in



FIG. 51.—W. G. HOYT



FIG. 52.—O. E. MEINZER

1892, were published posthumously in 1941 by ASME as a tribute to the memory of this distinguished leader.

⁴⁹ G. W. Rafter, Report on special water-supply investigation. Report of the Board of Engineers on deep waterways between the Great Lakes and the Atlantic tide waters, Part 2, pp. 571-950, pl. 93-130. Washington 1900, Congressional Documents 4146, 4147. Also: G. W. Rafter, On the flow of water over dams. Transactions, ASCE, 44(1900), Paper No. 884, pp. 220-314; discussions, pp. 315-398.

⁵⁰ R. E. Horton, Weir experiments, coefficients, and formulas. Water-Supply and Irrigation Paper, No. 150, Washington 1906, 189 p., 38 pl. Revised: No. 200, 1907, 195 p., 38 pl.

⁵¹ R. E. Horton, Turbine water-wheel tests and power tables. Water-Supply and Irrigation Paper, No. 180, Washington 1906, 134 p.

E. S. Cole was the promoter of the use of the pitometer in the "traverse" method: the instrument is introduced through a corporation cock and shifted from point to point along the diameter. Cole's pitometer has two opposite nozzles, and for a check can be turned around. A manometer gives the difference in pressure, from which the velocity is computed. The first Cole's pitometer was installed in the water main of Terre Haute, Ind., in 1896. In 1903, in cooperation with H. Flad, Cole developed a photo-pitometer. The fluctuations of the liquid column in the manometer were recorded on a shifting sensitive plate.

In 1908 a Swiss engineer H. Dufour constructed a valve with a packing gasket. A current meter at the end of a rod was introduced into the pipe under pressure and shifted across it from point to point. A similar method was used by R. M. Hosea in Colorado in 1910. A Price current meter was used in 48 in.

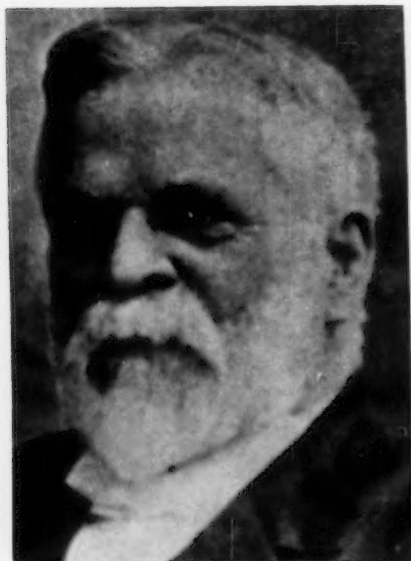


FIG. 53.—H. F. MILLS

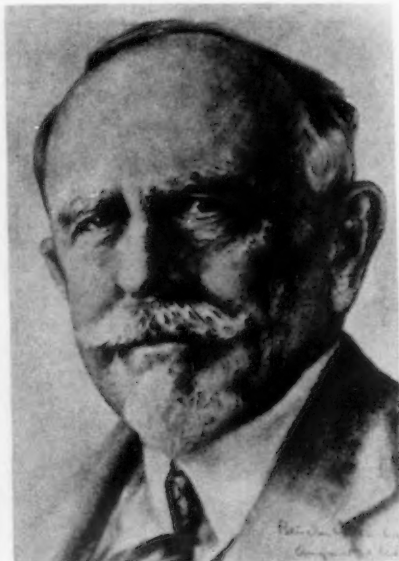


FIG. 54.—J. R. FREEMAN

and 28 in. wood-stave pipes of the Minnequa aqueduct. K. A. Heron performed similar work in 1914.

Several times current meters were adapted for the permanent control of flow. In 1898 A. Thiem installed a current meter of the Woltman type in a water main in Leipzig. It was equipped with a mechanism for transmitting the revolutions. In 1912 Hosea constructed a current meter of the cup type for a similar purpose. The vertical axle of the wheel was extended outside the pipe and connected to the counter.

Water meters for city supplies to householders were introduced after 1870, and now are manufactured and installed by the millions. The first water meter was patented in Britain by S. Crossley in 1825, in the United States by J. H. Hecker in 1848 and by W. Sewell in 1850, then by J. Ericsson in 1851. In 1855 H. R. Worthington invented the duplex piston-type water meter, in 1859 B. S. Church a rotary meter. In 1869 H. F. Read patented his gem-type velocity

meter, in 1879 L. H. Nash a crown-type meter, widely applied by the National Meter Co. John C. Kelley, the president of that pioneer company for 44 yr, was called the father of the water-meter industry in America. Up to 1890 as many as 678 patents were granted for water meters in the United States, 231 in Great Britain, and not less than 250 in other countries in Europe.⁵² The history of water meters is a subject of a special study.

MECHANICAL APPLIANCES

The technical skill of American inventors was displayed in numerous mechanisms designed for hydrometric purposes.

Water-stage recorders⁵³ with a wire transmission from a float and with a pencil marking a line on a clock-driven drum were constructed by Saxton in 1845, Trenchard in 1858, Avery in 1876, Fteley in 1876, Nettleton in 1884,



FIG. 55.—E. S. COLE

A. K. Warren in 1893, E. Mead in 1894, J. P. Friez in 1900, Haskell in 1903, Hanna in 1909. Since 1910 the firm of Barret and Lawrence produced a "hydrochronograph" which can run longer than a week. In 1912 Stevens designed a recorder with a continuous paper band sufficient for a whole year, having a weight-driven drum. Stevens also introduced a mechanism for a reversed motion of the pencil after it reaches the paper edge. In 1912 W. & L. E. Gurley in Troy, N. Y., developed a printing gage, which automatically printed the gage height on a paper strip every 15 min. This complex machine had little success,

⁵² J. Thomson, A memoir on water meters. Transactions, ASCE, 25(1893), No. 1, July, pp. 40-65. New York.

⁵³ It may be mentioned as a point of interest that before 1900 the water stage recorders or automatic gages were called nilometers, obviously as a tribute to the old Egyptian gages on the Nile River. In Europe the term limnigraph is in general use.

as also the efforts of J. M. Diven to introduce a pneumatic gage, in 1915. In 1920 Aud devised a practical recorder with a "positive drive", which was manufactured by Julien P. Friez Co. of Baltimore, Md., and widely used by the U. S. Geological Survey.

Long-distance recorders came into use in the 20th century.⁵⁴ In 1900 W. M. Fulton, professor of the University of Tennessee, devised a recorder with telegraphic transmission of the gage reading. In 1903 such a recorder was installed on the Tennessee River at Chattanooga and the receiver was located in the local weather bureau. The installation worked successfully.

Another device was important for mechanical conversion of gage readings into corresponding water discharges, and of their integration for continuous metering. This task was easy to carry out for the flow over a weir, where a definite relationship of certain power is known. A spiral cam of appropriate contour is used in the transmission mechanism. This principle was applied by



FIG. 56.—C. H. AU



FIG. 57.—J. W. LEDOUX

Herschel in 1888 and in Germany by Seibt in 1891, and was patented by D. L. Huntington of Scotland in 1896, in America by J. W. Ledoux in 1905, by Neibuhr in Germany in 1907, again in America in different forms by Hanna in 1910 and 1914, by Hosea in 1912, by Stevens in 1917. In 1912 D. R. Yarnall applied a drum with spiral groove. A hydrostatic transmission based on a form of an immersed paraboloidal body was invented by Ledoux, an engineer in Philadelphia, in 1913. The same idea was developed in Russia by V. G. Glushkov in 1916 and in Switzerland by J. Schenker in 1924.

⁵⁴ Electrical transmission of water stages was planned by W. Siemens in Germany in 1866, introduced by F. von Hefner-Alteneck in 1880.

Similar devices were constructed for venturi meters, so that the total quantity of water could be recorded without the continuous presence of an observer. In 1907 F. N. Connet and W. W. Jackson, of the Builders Iron Foundry, invented a register for venturi meters. In 1909 Connet introduced the square-root cam and a radial planimeter for continuous integration.

This problem is more complex in natural streams. Discharge integrators were invented for mechanical transformation of water stages into discharges according to an established empirical relationship. The idea of a mechanical device for transformation of the gage heights into water discharges was originated by W. L. Butcher in Cambridge, Mass., in 1905.⁵⁵ He designed a templet with a groove corresponding to the particular discharge curve. Another straight groove was shifted following the stage fluctuations. A polar planimeter, with a tracing pin connected to the curve, directly integrated the discharges. Simple devices for discharge transformation were proposed by F. Weber in 1914 and by F. H. Kingsbury in 1917. In 1915 E. S. Fuller designed and built a discharge integrator for the U. S. Geological Survey. This type, with some modifications, is still used. W. H. R. Nimmo in Australia, later the Water Commissioner in Queensland, designed a water discharge integrator in 1917. The first discharge integrator in Europe was constructed by Gocht and K. Rose in Saxonia in 1934.

It is proper to mention that the integrating float, a device emerging from the bottom of a channel and measuring an average velocity along a vertical, was also invented in America, in 1882. The inventor was Luigi d'Auria, who came from Italy in 1876. This ingenious device was revived by Hajos in 1904, and by Glushkov in 1909.

The tide-predicting machine was invented in England by Thomson (Lord Kelvin) in 1875 and improved by E. Roberts in 1879. In 1880 W. Ferrel, meteorologist of the U. S. Coast Survey, constructed a device for forecasting of extreme stages, height and time, called "Maximum and minimum tide prediction machine", set for the constants representing nineteen harmonic components of the tide. This machine was in use until 1910, when R. A. Harris and E. G. Fischer constructed a better tide predictor, which used 37 components.⁵⁶

CURRENT METER RATING STATIONS

Current meters require rating or calibration in still water which must be repeated from time to time. Formerly the rating was carried out from a boat pulled by a rope or by oars. A lake or a dead arm of a river was used for field rating. Several rating stations of a more permanent character were built during the early development of hydrometry in the United States:

- 1) In Denver, Colo., a water reservoir was used. A slot 16 in. wide, 200 ft long, was made in its roof. A car ran on rails along the slot, pulled by a cable on a drum.
- 2) In Kensington, Md., at Chevy Chase. A channel was dug, 165 ft long, over 5 ft wide, filled with water 4 ft to 12 ft deep. A car was pushed by hand.
- 3) In Lawrence, Mass., a wharf of the Essex Co., 200 ft long, was used for rating. A car was pushed by hand.

⁵⁵ W. L. Butcher, A device for averaging certain kinds of continuous records by the planimeter. *Engineering News*, 53(1905), No. 26, Jun. 29, p. 685. New York.

⁵⁶ A. J. Wraight and E. B. Roberts, *The Coast and Geodetic Survey, 1807-1957*, 150 years of history. Washington, 1957.

4) In Kern County, Calif., a pool was used. A wire was stretched across it, a roller was pulled along the wire with a current meter suspended on a cable.

Several rating arrangements were available at universities:



FIG. 58.—RATING STATION AT CHEVY CHASE, MD.

1) The naval tank of the University of Michigan at Ann Arbor, Mich., 360 ft long, 22 ft wide, 10 ft deep.

2) The Hydraulic laboratory of the Rensselaer Polytechnic Institute at Troy, N. Y., has a rating channel 90 ft long, 3.75 ft wide and 3.5 ft deep. A hand-propelled car regulated by a tachometer. An electric chronograph installed.



FIG. 59.—RATING STATION IN CALIFORNIA

3) The Alden Hydraulic laboratory of the Worcester Polytechnic Institute, Mass. has a rating installation of a circular type on a pool. In 1908 a rotating beam was arranged, which is moved by a belt from a water turbine. The diameter of the circle run by the current meter is 84 ft.

HYDROMETEOROLOGICAL MEASUREMENTS

Observation of hydrometeorological elements, precipitation and evaporation, is usually carried out by the meteorological organizations. The earliest

precipitation records in the United States were started in New Haven, Conn., in 1804; in Brunswick, Me., in 1808; in Spring Mill, Pa., in 1810. In 1825 the Medical Department of the U. S. Army organized a network of stations for recording of temperature and precipitation. The Smithsonian Institution supervised the activity of that network from 1849 until 1891, when the Weather Bureau was established in the Department of Agriculture.

Rain gages, ombrometers or pluviometers, were used of various forms: a conical gage with 5 in. circular opening, then cylindrical of 11.3 in. diameter. In 1824 G. Chilton designed a rain gage with a rectangular orifice of 10-in. square. Later on a circular orifice of 8 in. in diameter was approved as a standard. A windshield was tried by J. Henry in Smithsonian Institution, in 1853; a horizontal brim, 1 in. below the orifice, was used to protect raindrops from diversion. In 1878 F. E. Nipher, Professor in St. Louis, Mo., constructed an inverted cone to protect the snow catch against the wind suction. The modified Nipher shield was used in different countries.

The first recording rain and snow gage, on weighing principle, was patented by S. P. Fergusson in 1889. Other recording gages were designed by Friez and by Stevens. The first precipitation map with isohyets was published by L. Blodget in 1855. More detailed "rain charts" were made by C. A. Schott in 1873 and 1881.

Observations of the evaporation from pans were developed in more recent time.

PIONEER WORK OF THE ACADEMIC STAFF

A significant fact cannot be overlooked: the outstanding role of the American professors in the early history of the hydrometry. A majority of the distinguished men in this field, as Forshey, Powell, Newell, Grover, Murphy, Barrows, Haskell, Shenehon, Stewart, and Hunt, were faculty members. Many professors of civil engineering dedicated their summer vacations to the hard field work as part-time hydrographers with payment of \$5.00 a day when actually employed. Stout, S. Fortier, L. G. Carpenter, B. M. Hall, D. C. Humphreys, T. U. Taylor, J. A. Holmes, A. M. Ryon, C. N. Brown and others took part in the development of the U. S. Geological Survey.

Often university professors performed a pioneer work in measuring the flow of streams with their students as a field practice. G. H. Hamlin of Main State College measured discharge of the Penobscot River: in 1884 he used subsurface floats, in 1886 the Ellis current meter. These measurements were the first in the State of Maine. D. Porter, of the Massachusetts Institute of Technology, measured with his students the Delaware River in 1891. Similarly, C. H. McLeod, Professor of the McGill College in Montreal, Canada, used rod floats and a current meter for the discharge measurement of the St. Lawrence River, 40 miles below Montreal, in 1895.

The work of D. C. Humphreys, Professor of the Washington and Lee University at Lexington, Va., must be particularly commended. In 1897, together with F. H. Anschutz and W. A. Shepherd, he arranged an expedition for investigation of the entire James River course in Virginia. They traveled in canvas canoes, each paddling his own boat. A Haskell meter on a wooden rod was used for discharge measurements by wading or from a canoe. At night the boats were lifted out on the bank and the tents erected for sleeping. The canvas

canoes were protected from rocks by outside strips and proved to be very effective. Two months were spent in this successful expedition.⁵⁷

CONCLUSIONS

Space limits abridged this study to the early times. There are no rigid bounds in the progress of science, therefore the approximate end of this period was assumed as World War I.

The era of the modern developments in hydrometry may be landmarked with the penetration of automobile into American life. This resulted in the construction of cableways for one-man operation, the power-drive for heavy current meter, comfortable ships for navigable rivers, automation and transmission of observations, power plant testing by the Allen and the Gibson methods,



FIG. 60.—JAMES RIVER 1897 EXPEDITION CAMP

measurements in penstocks and in turbine inlets, the ingenious flumes for irrigation canals, modern instrumentation for data processing, finally the well developed laboratory devices and the new principles of measurements. All this is to be a topic for future continuation of this subject.

APPENDIX I.—HYDROMETRY

Hydrometry is a part of hydrology. It deals with water measurements. This term is known in all European languages—Bulgarian, Czech, Danish, Dutch,

⁵⁷ F. H. Newell, Report of progress of stream measurements for the calendar year 1897. In the 19 annual report of the U. S. Geological Survey, IV, Hydrography. Washington 1899, pp. 162-172.

Estonian, Finnish, Flemish, French, German, Hungarian, Italian, Latvian, Lithuanian, Polish, Portuguese, Rumanian, Russian, Serbo-Croatian, Slovak, Slovenian, Spanish, Swedish and Ukrainian. It is disliked in English and usually replaced by hydrography, stream gauging or gaging.⁵⁸

Hydrometry was earlier well known in America. In 1851 three surveying parties were organized on Lower Mississippi: topographical, hydrographical, and hydrometrical. The Organic Act of 1879 of the U. S. Congress created the Mississippi River Commission and authorized it "to make surveys, examinations and investigations, topographical, hydrographical, and hydrometrical, of said river and its tributaries". D. W. Mead, author of the first American systematic treatise "Notes on Hydrology", 1904, entitled one chapter "Hydrometry". In his fundamental work "Hydrology", however, this chapter has been called "Stream flow or runoff". An article "Hydrography as an aid to the successful operation of an irrigation system" by Stevens was published in the Proceedings, ASCE, in 1911. Herschel requested in discussion to correct the title, and this paper was reprinted in the Transactions, ASCE, in 1911, under a title "Hydrometry as an aid to the successful operation of an irrigation system".

A conference of district engineers of the U. S. Geological Survey in Washington, D. C., in 1911, adopted a resolution favoring the use of the term hydrometry instead of the previous hydrography. Regrettably, this decision was not followed up by the office. Therefore instead of books on hydrometry they are on "river discharge", "stream gaging", "stream flow", "flow measurement".

A source of disgrace to hydrometry in England and in America can be traced to the misuse of this term by physicists for alcoholometry or ebulliometry. Dictionaries explain hydrometry as the art of using the hydrometer, a device for determining the properties of liquids, such as density, specific gravity, purity, and, particularly, the strength of spirits; such an instrument is called in all languages araeometer or ebulliometer. There are publications on that particular "hydrometry", e.g., "Handbook



FIG. 61.—D. W. MEAD

⁵⁸ It is interesting to call attention of readers to a definition of the hydrometry in an old collection—The Cyclopaedia, or universal dictionary of arts, sciences, and literature, edited by Abraham Rees in London, in 39 volumes, in 1819:

Hydrometria, or hydrometry, the mensuration of water and other fluid bodies, their gravity, force, velocity, quantity, etc. Hydrometria includes both hydrostatics and hydraulics. The term is modern but very little in use. The first instance where we meet with it, is in the year 1694, when a new chair, or professorship of hydrometry was founded in the university of Bologna, in favour of S. Guglielmini, who had carried the doctrine of running waters, with respect to rivers, canals, dykes, bridges, etc., to an unusual length.

of Hydrometry", by J. B. Keene, London 1875, "Hydrometers and Hydrometry", by E. R. Crandall, Detroit 1954.

The author suggests use of the following logical terminology:

Hydrometry—science of water measurements, instead of hydrography,

Hydrometer—instrument for water measurement,

Hydrometric engineer or hydrometrists, instead of hydrographer,

Hydrometric Bureau—a central office and local districts,

Hydrometric station, instead of gaging station, discharge range,

Hydrometric yearbook, instead of Stages and Discharges, Water-Supply Papers, Daily river stages, etc.

APPENDIX II.—SHORT BIOGRAPHICAL DATA

Henry Larcom Abbot, b. Aug. 13, 1831, at Beverly, Mass., d. Oct. 1, 1927, in Washington, D. C.; 1854 grad. West Point, 1857-61 Mississippi Survey, 1895 Brig. Gen., 1898-1915 Panama Canal, cons. eng.

Zachariah Allen, b. Sept. 15, 1795, in Providence, R. I.; d. Mar. 17, 1882, also in Providence; 1813 grad. Brown Univ., engineer and inventor.

Carl Henry Au, b. Aug. 3, 1876, in Washington, D. C., d. Nov. 24, 1958, also in Washington; grad. Worcester Polytech. Inst., instr. 8 years, 1916-31 U. S. Geolog. Survey, Chief of Instr. Branch.

Alexander Dallas Bache, b. Jul. 19, 1806, in Philadelphia, Pa., a great-grandson of Benjamin Franklin, d. Feb. 17, 1867, in Newport, R. I.; 1825 grad. West Point, 1828-41 Prof., Univ. of Pennsylvania, 1843-67 U. S. Coast Survey, Superintendent, first Pres. National Academy of Sc.

James Fowle Baldwin, b. Apr. 29, 1782, at North Woburn, Mass., d. May 30, 1862, in Boston, Mass.; stud. Wertford Acad., 1830-35 constr. railroad, 1837-48 Boston waterworks.

Harold Kilbreth Barrows, b. Nov. 8, 1873, in Melrose, Mass., d. Mar. 15, 1954, at Winchester, Mass.; 1895 grad. Mass. Inst. Tech., 1895-1904 Univ. Vermont, 1904-08 U. S. Geolog. Survey, 1908-41 Prof. Mass. Inst. Tech.

David Stanhope Bates, b. Jun. 10, 1777, near Morristown, N. J., d. Nov. 28, 1839, in Rochester, N. Y.; 1817-23 constr. aqueducts and canals.

Murray Blanchard, b. Jul. 25, 1874, in Peru, Ill.; 1898 grad. Univ. Mich., 1899-1905 U. S. Lake Survey, 1919-56 Chicago waterworks.

Roy Hale Bolster, b. Apr. 6, 1877, in Roxbury, Mass.; 1901 grad. Mass. Inst. Tech., 1903-07 U. S. Reclam. Bureau, 1907-22 U. S. Geolog. Survey.

Edward Smith Cole, b. Dec. 29, 1871, in Washington, D. C., d. Mar. 18, 1950, at Upper Monclair, N. J.; 1894 grad. Cornell Univ., Pitometer Co. Director.

Frederick Nevius Connet, b. Oct. 16, 1867, at Flemington, N. J., d. Jun. 18, 1935, in Providence, R. I.; 1889 grad. Stevens Inst. Tech., 1889-1930 Chief eng., Builders Iron Foundry; more than 30 patents.

Charles Ellet, Jr., b. Jan. 1, 1810, at Buck's Manor, Berks County, Pa., d. Jun. 21, 1862, at Cairo, Ky., injured in naval battle; 1834 grad. Ecole Polytechnique, Paris, France, 1834-49 bridge construction, 1850-53 Ohio River.

Theodore Grenville Ellis, b. Sep. 25, 1829, d. Jan. 8, 1883, in Hartford, Conn.; 1854-61 constr., 1862 in Civil War, Brig. Gen., 1867-78 Connecticut R.

Samuel Forrer, b. Jan. 17, 1793, in Dauphin County, Pa., d. Mar. 25, 1874, in Dayton, Ohio; 1820 Ohio State eng.

Caleb Goldsmith Forshey, b. Jul. 18, 1812, in Somerset County, Va., d. July 25, 1881, in New Orleans, La.; 1835 grad. West Point, 1837-57 surveyor,

1853-61 Prof. Mil. Acad. Texas, 1862 Confederate Army, Lt. Col., Vice Pres. New Orleans Acad. Sc.

James Bicheno Francis, b. May 18, 1815, in Southleigh, England, 1833 in U. S. A., d. Sep. 18, 1899, in Boston, Mass.; 1837-85 Merrimac Co. eng., Lowell Hydraulic lab.

John Ripley Freeman, b. Jul. 27, 1855, in West Bridgeton, Me., d. Oct. 6, 1932, in Providence, R. I.; 1876 grad. Mass. Inst. Tech., 1876-1932 cons. eng.

Alphonse Fteley, b. Apr. 10, 1837, in Paris, France, 1865 in U. S. A., d. Jun. 11, 1903, in Yonkers, N. Y.; 1875-84 Boston waterworks, 1884-1900 New York waterworks, 1898 ASCE Pres.

Nathan Clifford Grover, b. Jan. 31, 1868, in Bethel, Me., d. Nov. 29, 1956, at Washington, D. C.; 1890 grad. Univ. Maine, 1891-1903 Prof. Univ. Maine, 1903-39 Chief eng. U. S. Geolog. Survey.

Carl Ewald Grunsky, b. Apr. 4, 1855, in San Joaquin County, Cal., d. Jun. 9, 1934, in San Francisco, Cal.; 1877 grad. Tech. Hochschule Stuttgart, Germany, 1875-1905 eng. in Cal., 1904-05 Panama Canal, 1908-34 cons. eng., 1924 ASCE Pres.

Frank Willard Hanna, b. Sept. 16, 1867, near Geneseo, Ill., d. Jan. 26, 1944, in Webster City, Ia.; 1893 grad. Des Moines Jun. Coll., Ia., 1903-05 U. S. Geolog. Survey, 1905-21 U. S. Reclam. Bureau, 1921-34 in Canada.

Eugene Elwin Haskell, b. May 10, 1855, in Town of Holland, Erie County, N. Y., d. Jan. 29, 1933, in Hamburg, N. Y.; 1879 grad. Cornell Univ., 1893-1906 U. S. Lake Survey, 1906-21 Prof. Cornell Univ., Dean of Engg.

Daniel Farrand Henry, b. May 27, 1833, in Detroit, Mich., d. May 13, 1907, also in Detroit; 1853 grad. Yale Univ., 1853-71 U. S. Lake Survey, 1872-78 Chicago waterworks, 1880 cons. eng.

Clemens Herschel, b. Mar. 23, 1842, in Vienna, Austria, grown in Davenport, Ia., d. Mar. 1, 1930, in Glen Ridge, N. Y.; 1860 grad. Harvard Univ., 1864 eng. in Boston, 1872 Holyoke Power Co., 1900 cons. eng., 1916 ASCE Pres.

Robert Elmer Horton, b. May 18, 1875, at Parma, Mich., d. Apr. 22, 1945, in Voorheesville, N. Y.; 1897 grad. Albion Coll., Mich., 1900-11 U. S. Geolog. Survey, 1911 cons. eng.

John Clayton Hoyt, b. Jun. 10, 1874, in Lafayette, N. Y., d. Jun. 21, 1946, in Paris, Va.; 1897 grad. Cornell Univ., 1902-44 U. S. Geolog. Survey.

William Glenn Hoyt, b. Aug. 25, 1886, in Lafayette, N. Y.; 1909 grad. Cornell Univ., 1907-44 U. S. Geolog. Survey, 1944-50 Water and Power Div.

Andrew Atkinson Humphreys, b. Nov. 2, 1810, in Philadelphia, Pa., d. Dec. 27, 1883, in Washington, D. C.; 1831 grad. West Point, 1851-60 Mississippi Survey, 1861-64 in Civil War, Brig. Gen., 1866-79 Chief Engrs.

Edward Bissell Hunt, b. Jun. 15, 1822, at Portage, Livingston County, N. Y., d. Oct. 2, 1863, at New York City; 1845 grad. West Point, 1846-49 Prof. West Point, 1834-62 U. S. Coast Survey.

Joseph Christmas Ives, b. 1828, in New York City, d. Nov. 12, 1868, also in New York City; 1852 grad. West Point, 1857-58 explor. Colorado River, 1861 in Confederate Army, Col., Chief eng.

John Bloomfield Jervis, b. Dec. 14, 1795, at Huntington, L. I., d. Jan. 12, 1885, in Rome, N. Y.; 1817-25 constr. Erie Canal, 1825-30 Chenango Canal, 1836-46 Croton aqueduct.

John Walter Ledoux, b. Aug. 28, 1860, at St. Croix Falls, Wis., d. Nov. 7, 1932, at Media, Pa.; 1887 grad. Lehigh Univ., 1891 Amer. Pipe Manuf. Co. eng., 1905 Simplex Valve and Meter Co.

Alexander Mackenzie, b. May 25, 1844, in Potosi, Wis., d. Feb. 23, 1921, in Washington, D. C.; 1864 grad. West Point, 1866-95 Mississippi River, 1895-1908 Chief Engrs.

Daniel Webster Mead, b. Mar. 6, 1862, in Fulton, N. Y., d. Oct. 13, 1948, in Madison, Wis.; 1884 grad. Cornell Univ., 1904-32 Prof. Univ. Wisconsin.

Oscar Edward Meinzer, b. Nov. 28, 1876, near Davis, Ill., d. Jun. 14, 1948; 1901 grad. Beloit Coll., 1906 U. S. Geolog. Survey.

Hiram Francis Mills, b. Nov. 1, 1836, at Bangor, Me., d. Oct. 4, 1921, at Hingham, Mass.; 1856 grad. Rensselaer Polytech. Inst., 1896 Chief eng. Essex Co., Lawrence, Mass.

Henry Mitchell, b. Sep. 16, 1830, in Nantucket, Mass., d. 1902, also in Nantucket; 1867 grad. Harvard Coll., 1850-88 U. S. Coast Survey.

Edward Charles Murphy, b. Jun. 17, 1859, in Croydon, Ont., Canada, d. Sep. 18, 1934, in Mar Vista, Cal.; 1884 grad. Cornell Univ., 1887 Univ. Kansas, 1892-1902 U. S. Geolog. Survey, 1901-02 Prof. Cornell Univ.

Frederick Haynes Newell, b. Mar. 5, 1862, in Bradford, Pa., d. Jul. 5, 1932, in Washington, D. C.; 1885 grad. Mass. Inst. Tech., 1898-1902 U. S. Geolog. Survey, 1902-14 U. S. Reclam. Bureau, 1914 Prof. Univ. Illinois.

John Wesley Powell, b. Mar. 24, 1834, at Mount Morris, N. Y., d. Sep. 23, 1902, in Haven, Me.; 1859 stud. Oberlin Coll., 1861-65 in Civil War, Major, 1865-72 Prof. Illinois Wesleyan Univ., 1881-94 U. S. Geolog. Survey, Director.

William Gunn Price, b. Jul. 6, 1853, in Knoxville, Pa., d. Jul. 6, 1928, in Detroit, Mich.; grad. Columbia Univ., 1879-96 Mississippi River, 1896-1913 Chicago City eng.

George Willson Rafter, b. Dec. 9, 1851, in Phelps, N. Y., d. Dec. 29, 1907, in Karlsbad, Austria; grad. Cornell Univ., 1894 New York State eng.

John Leonard Riddell, b. Feb. 20, 1807, in Leyden, Mass., d. Oct. 7, 1863, in New Orleans, La.; 1829 grad. Rensselaer Polytech. Inst., 1836 Prof. Louisiana Med. Coll.

Louis Carlton Sabin, b. Jun. 25, 1867, in Memphis, Mich., d. Dec. 30, 1950, in Cleveland Heights, Ohio; 1890 grad. Univ. Mich., 1899-1902 U. S. Lake Survey, 1906-25 constr. St. Mary's Falls Canal.

Joseph Saxton, b. Mar. 22, 1799, at Huntington, Pa., d. Oct. 26, 1873, in Washington, D. C.; 1830-37 in England, 1837 mech. U. S. Coast Survey.

Francis Clinton Shenehon, b. Dec. 20, 1861, in Brooklyn, N. Y., d. Oct. 3, 1939, in Minneapolis, Minn.; 1895 grad. Univ. Minn., 1898-1908 U. S. Lake Survey, 1909-1917 Prof. Univ. Minnesota, Dean of Engg.

Frederic Pike Stearns, b. Nov. 11, 1851, at Calais, Me., d. Dec. 1, 1919, in Boston, Mass.; 1872-86 Boston waterworks, 1886-1907 N. Y. Metropolitan Water Board, 1906 ASCE Pres.

John Cyprian Stevens, b. Jan. 9, 1876, in Moline, Kans.; 1905 grad. Univ. Nebr., 1903-05 U. S. Geolog. Survey, 1905 cons. eng., Portland, Ore., instr. constr., 1945 ASCE Pres.

Clinton Brown Stewart, b. Mar. 8, 1868, in Fairbury, Ill., d. Sept. 2, 1951, in Anchor, Ill.; 1890 grad. Cornell Univ., 1893-98 U. S. Lake Survey, 1899-1900 Univ. Wis., 1900-03 Prof. Colorado School of Mines.

Charles Storer Storrow, b. Mar. 25, 1809, in Montreal, Canada, d. Apr. 30, 1904, in Boston, Mass.; 1829 grad. Harvard Univ., 1829-31 Ecole des Ponts et Chausees, Paris, France, 1844-89 Essex Co., Merrimac River.

Oscar Van Pelt Stout, b. Nov. 14, 1865, in Jerseyville, Ill., d. Aug. 4, 1935, in Berkeley, Cal.; grad. Univ. Nebr., 1891-1929 Prof. Univ. Nebraska, Dean of Engg.

Andrew Talcott, b. Apr. 20, 1797, in Glastonbury, Conn., d. Apr. 22, 1883, in Richmond, Va.; 1818 grad. West Point, 1837-39 Mississippi River, astronomer.

George Washington Whistler, b. May 19, 1800, in Fort Wayne, Ind., d. Apr. 7, 1849, in St. Petersburg, Russia; 1819 grad. West Point, 1821-22 Ass. Prof. West Point, 1833 Major, resigned, 1828-33 railway constr., 1833 Lowell, Mass., locks and canals, 1842 cons. eng. in Russia.

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Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

NEW APPROACH TO LOCAL FLOOD PROBLEMS^a

By Herbert D. Vogel,¹ F., ASCE

SYNOPSIS

Paper describes the growing pressures of population and urban growth tending to increase encroachment on the flood plain, thereby increasing flood damage potential. Reviews TVA experience in identifying flood-danger areas of cities and encouraging local action to guide urban development away from these areas.

Four months ago the Tennessee Valley Authority sent to Congress a special report in which it said in part:

Communities throughout the Nation are engaged in a new contest with their rivers, and they are losing. They will continue to lose unless steps are taken to provide a new perspective—and a new channel of action—with respect to floods.

Last May, Brig. Gen. John L. Person, Asst. Chief of Engineers for Civil Works, made the following remarks in testimony before the House Committee on Public Works:

While we have made great progress in providing flood control works, many of our river valleys are still subject to destructive floods, and the degree of protection varies widely. Moreover, it will probably not be possible, because of physical and economical limitations, to provide full

Note.—Discussion open until June 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 1, January, 1960.

^a Presented at the July, 1959, ASCE Hydraulics Division Conference in Fort Collins, Col.

¹ Brig. Gen. (U. S. Army Ret.), Chmn., Board of Directors, TVA, Knoxville, Tenn.

flood protection. This leads to the inescapable conclusion that greater attention must be given by states, municipalities, and industry, and by the federal agencies concerned with development, to some form of regulation of flood plain use . . . We should be as much concerned with avoidance of creating a future flood hazard, as with means of correcting the damage after it occurs.

About six months ago the Council of State Governments, following a conference of the General Assembly of the States in Chicago, issued a statement of its conclusions reading as follows:

Each state should promptly review its existing legislation and administration to determine what steps are needed to authorize the use of zoning, subdivision regulation, building codes and other means of land use regulation to prevent flood losses.

Behind these three statements is the fact that engineers and public officials concerned with our rivers and their development are taking a new look at an old problem—the problem of preventing the damage, the suffering and the economic dislocation caused by swollen, flooding streams. People concerned with the problems of expanding urban communities are realizing that all is not being done that can be done to prevent damage from floods.

Behind this realization is a background of dynamic trends in our national life. The 50-year record of flood damages since 1902, shows that in spite of all our Nation's effort at flood control, the average annual losses have increased rather than decreased, even when adjusted to the changes in the value of the dollar. Even more significant, the losses during the last half of this period were twice as great as those in the first half.

The difficulty is not that we have not built well. Some of the world's finest engineering achievements in flood control are right in this country. Rather, the difficulty lies in the ever-shifting, ever-changing nature of the use of the flood plain as the growing population moves about in response to altered social and economic conditions. The United States is no longer an agricultural nation. The people now live in an urban-industrial economy. Between 1900 and 1950, the number of urban places, as defined by the U. S. Census, has increased from 1,737 to 4,023. Since 1930 alone, the population of the United States has increased by 52 million, and nearly all of the increase has occurred within urban areas and on their fringes. Current evidence indicates that the population in 1975 will be some 55 million greater than today. And again, almost all of this growth will be in and around our cities and towns.

These forces create powerful and irresistible demands for lands for homes, shopping centers, and commercial and industrial plants, spreading and radiating far beyond the formal boundaries of present cities. In many cases, the most attractive land, the land that seems subject to development most economically, is the property in the level river valleys where the flood hazard is greatest.

In the past five years alone, a dozen or more states have felt the serious and damaging consequences of flood-plain development. Disastrous events have occurred in rivers located throughout the country and particularly in Connecticut, Massachusetts, New York, Pennsylvania, Indiana, Illinois, Texas, Kentucky, Kansas, California, Ohio, and Missouri.

Along the rivers and along the low-lying coastal areas, population pressures have brought a steady increase in the flood-damage potential. These are the areas vulnerable to the abnormal tides and waves pushed on shore by hurricanes, which batter and flood the structures built too close to the sea. Disasters which received national notice occurred in Rhode Island in 1954, North Carolina in 1955, and Louisiana in 1957.

Communities faced with population pressures are confronted at the same time with forces pressing upon them from another direction. With a national government hard pressed to meet the financial requirements of defense and the bolstering of the free world, and with state governments pushed to meet the essential expenditures required for such vital programs as education and highways, the funds available for protective structures along our rivers have been inadequate to keep pace with the growth of areas needing protection. At present rates of construction, projects now authorized cannot be completed in less than 25 to 30 years. In the meantime new areas will be built up and new floods will descend to destroy them—unless steps are taken to avoid this result. Time works against us. No matter how energetically we build, we fall further behind in the task.

This is a situation which, by its very nature, the engineer understands best. It is therefore a situation in which engineering leadership in its broadest sense is urgently required. The country is faced with a combination of physical facts which add up to a growing peril for valley-dwelling people and to a growing financial burden for the Federal Government. It is an obligation, therefore, of the engineering profession to make it clear far and wide that a new approach to flood problems is essential, and that time is of the essence getting new programs in operation.

As TVA specifically recommended in its report to Congress, as General Person stated in his recent testimony, and as the Council of State Governments concluded after a conference on the flood problem—the answer lies in the recognition by states and municipalities of a new conception of their responsibilities, which embrace preventive as well as corrective measures. In the initiation and exercise of those responsibilities, engineers have a primary role.

The modern sciences of meteorology and hydrology make it possible to determine with fair precision the kinds of storms a region may expect, the volume of water that may fall as rain, and the rate of speed at which it will descend from the hills and mass itself with its titanic force in the flood plains below. Engineers can determine the areas the water will cover. The only element of conjecture in their analysis is the timing. These great floods can come any time; within 10 years or not for a hundred years. They may come this year and next year too.

With these great sciences at our command, we must not limit our flood damage avoidance programs to control measures alone. Modern scientific methods can help guide the surging growth of urban areas to grounds that are safe from floods; thus, the flood problem can be remedied in large part by keeping people out of the pathway of the waters. As foresters have learned to exert their primary efforts toward preventing fires, rather than suppressing them once they have started, engineers can educate America to this new truth with respect to floods—that flood damage avoidance goes hand in hand with flood control.

In the light of these clear implications, it is worth while examining the experience of the T.V.A. in avoiding damage from floods in the Tennessee

Valley. Ordinarily such floods cannot be controlled except by extremely large, and probably unwarranted, expenditures. It is probable that the techniques tested there over the last six years are adaptable to other states and localities; that their widespread use can save lives and property beyond any ability to estimate and can at the same time lessen the pressures on the Federal Treasury for local flood-control construction.

Historically, our attempts as a Nation to solve the flood problem have been directed almost exclusively to the use of flood-control structures. This was a natural development because early settlements were tied closely to the rivers for economic reasons. As early as 1717, plantation owners built levees along the banks of the Mississippi at New Orleans, and in the next 100 years both banks of that great river were leveed for over 300 miles. States and local communities followed this pattern. So did the Congress in passing the Federal Flood Control Act of 1936 and its subsequent amendments, under which most of the flood control work of the Nation is now undertaken. TVA's system of storage dams and of regulating those dams followed in this pattern: They construct artificial works to master the extreme flows of the streams.

To the degree for which they were designed, works such as these protect existing investment in many industrial centers vital to the economy. They, thus, produce major benefits. In the Tennessee Valley, dams and reservoirs provide flood protection to more than a quarter of a million acres within the Valley and to 4 million acres of unleveed land in the lower Ohio and Mississippi Valleys. Complete protection has been afforded in some areas, while in others the depth and frequency of flooding have been materially reduced. The effectiveness of TVA's flood-control system is well established. Average annual benefits amount to \$13 million. Actual flood damages averted to date by the reservoir system (\$140 million) plus the increase in land values resulting from the greater security it provides to leveed areas along the Mississippi (\$150 million) are already more than sufficient to amortize the entire investment in flood-control facilities (\$184 million) plus all operating expenses and an allowance for interest (\$84 million).

Few systems of river regulation provide complete protection. The Tennessee Valley can still have floods, even disastrous floods. The greatest danger is at Chattanooga, Tennessee. TVA has warned time after time of the need for levees and for the regulation of the use of the flood plain to supplement the protection provided by the TVA reservoir system. Unless Chattanooga builds these levees and establishes the needed regulations it is certain that the city one day will suffer a \$100,000,000 disaster.

Furthermore, there are many communities on small tributary streams for which there is no economic method of preventing floods. Dams or levees would cost as much—or more—than the property in need of protection. Athens, Tennessee, on Oostanaula Creek; Gatlinburg, Tennessee, the Great Smoky Mountain resort center on Little Pigeon River; and Lewisburg, Tennessee, on Big Rock Creek, are examples of Valley communities, numbering perhaps 100, which can suffer crippling damage from severe local storms over their small watersheds.

In 1953, the TVA Board of Directors took cognizance of these local flood situations to establish a special branch to deal with local flood problems. In its annual report to the Congress that year, the Board said the following:

In the Tennessee Valley, as in other parts of the Nation, little initiative has been taken by states and cities in assuming responsibility for local

flood control problems. . . . There is much that communities, counties, and states could and should do for themselves in these local flood situations. In many cases the construction of protective works is within the financial capability of the communities involved. . . . Zoning by the state, county, or municipal governments against certain types of construction in hazardous areas is a device which needs further consideration and which may deserve much wider application. Effectiveness of zoning would be increased if there were less chance to pass the responsibility for these problems to the Federal Government.

This was the beginning of TVA's concerted effort to encourage and assist Valley communities to adapt to their flood danger on their own initiative, to guide their community growth so as to avoid the vulnerable flood plain areas. TVA has one important asset to offer as an incentive to these communities. This asset is a large pool of hydrologic data, gathered over many years and consisting of the records of rainfall, runoff, streamflow, flood profiles, and other factors which enable the engineers managing the TVA flood control system to predict with accuracy the effects of the violent rains which sometimes sweep the region. With a relatively small amount of supplemental field work and a small administrative staff, TVA can and does assemble all the pertinent information concerning the flood dangers of a particular city. It makes available to state and local officials reports couched in terms a layman can understand.

These reports include maps, profiles, and cross-sections clearly outlining the elevations of past floods and the areas inundated. Similar information is furnished concerning the larger floods that may be expected. The rate of rise and fall of flood waters as well as velocities in the main channel and in overflow areas is given because it indicates the time available for evacuation or protective measures and provides guides for the safe engineering design of structures.

Communities often regard their highest floods of the past as the greatest likely to recur. But often much more severe floods have occurred on other streams in their regions. Since it is reasonable to expect floods of about the same severity as those which have occurred in the general region, TVA makes studies and determines discharges for a regional flood; that is, the flood based on records of the large floods and storms that have occurred in the immediate region of the subject stream. TVA also determines the discharge for the still larger "maximum probable flood." These studies provide the practical and useful framework necessary in developing an economical and reasonable solution to the flood problem.

Similar, though perhaps not as detailed, hydrologic and flood data are available today or readily obtainable for hundreds of areas throughout the Nation as a result of many years of work by other federal agencies. The Army Engineers, the Bureau of Reclamation, the Geological Survey, the Weather Bureau, and the Department of Agriculture have had hydrologists in this field for a long period and possess the information, skill, and experience needed to project this type of program to a national scale.

In the Tennessee drainage basin, TVA supplemented this engineering flood study with its very small community planning staff which keeps in constant touch with the planning agencies of the seven Tennessee Valley states as well as with local planning units.

The program within the Tennessee Valley achieved momentum in a surprisingly short time. Flood reports have been prepared for 53 communities, and reports for the next year will cover about 14 more. Communities are requesting flood data faster than it can be supplied.

In making the information available to local communities, TVA has the following three points of policy:

First, the initiative for a flood study must come from the local community, expressed officially by its elected officials.

Second, the local request must be sent to the appropriate state officials, who then must request the study for the state and community. The state has a heavy responsibility in carrying out these measures, and the T.V.A. does not short-circuit the state offices. At the same time, it is made plain to local officials that the facts obtained in a flood survey will be presented without sugar-coating and that changes will not be made in them to safeguard anyone's feelings or interests.

Third, the reports made no recommendations as to what the community shall do to meet its flood problem. The report is purely factual. It provides the base from which the state and local people can analyze their problems and plan to meet them. It is the logical first step in any program of flood-damage abatement. The data are needed for subsequent feasibility studies both for flood-control protective works and as a basis for determining flood-plain regulations. They will be equally necessary in supporting the reasonableness of such regulations if the regulations are contested in court.

The indispensable keys to the success of the program are effective action-agencies, usually planning agencies, at both the state and local level. In fact, no action occurs without them. The local community has the power to adopt standards for the use of flood-hazard areas and otherwise guide the use of such areas. Zoning, subdivision standards, building and housing codes, the location of schools, the construction of roads and bridges, the location of public utilities—all these are factors involving local jurisdictions and state laws, which only local and state units of government can deal with promptly and effectively.

Once the flood data are available the community is in a position to consider within the context of its general planning program alternative solutions to the flood problem. Where the land is already intensively built up, investigations would include the economic feasibility of physical flood control or protective works either as a separate project or as part of an urban renewal program. Where sparsely developed or open land exists, regulations covering the use of the flood hazard areas are considered. Both, however, would be related to the community's plan for development.

Frequently, the community involved is not financially able to maintain the staff of experts in the fields of engineering, law, housing, industrial development and planning which would enable them to utilize the findings of the flood control study effectively. An important responsibility of state governments is to fill this gap, to place at the disposal of the local communities a staff of specialists able to give technical assistance to small communities in need of such help.

Parenthetically, the engineering sciences have in recent years helped make it feasible to locate businesses and industries on sites well removed from the river bank. Electric power transmission moves the energy of the river over

many miles. In some instances the need for river-site locations can be eliminated through the use of conveyor belts and pipelines for transporting raw materials brought in over the waterway. Telephone and radio communication make direct access to the river for communication purposes unnecessary, and pipelines and pumps make it possible to bring industrial water considerable distances to processing plants.

The city of Lewisburg, in Tennessee, is a good example of a community which has responded to a TVA flood report with an action program of a comprehensive nature. Following receipt of TVA's flood data, the local Planning Commission, with the aid of the state planning agency and technical assistance from TVA, made a study to determine the best solution for its flood problem. Upon the Commission's proposal, the City Council revised the city's zoning ordinance to include flood-plain regulations. A floodway for the stream was designated. Within this floodway, structures that would block or unduly interfere with flood flows were prohibited. The floodway was not to be unused, however; parking lots, outdoor theaters, and similar facilities not subject to severe damage were permitted. Buildings along the fringes of the floodway were required to have floor levels at an elevation which would be above the flood stage for which zoning was adopted.

TVA's flood data reached Lewisburg in time to prevent the opening of two residential subdivisions which had been laid out in areas subject to frequent inundation. Without this data, the subdivisions almost certainly would have been approved and 20 or more houses could have been completed by the time of the March 1955 flood, which would have put four feet of water over their floors.

Other cities have had an opportunity to make similar planning adjustments as a result of an accurate knowledge of their flood problem. City officials of Cleveland, Tennessee, expanded their purchase of a school site to include flood-free land suitable for building. The land which is subject to flooding was devoted to playground use. Chattanooga, Dayton, and Spring City, Tennessee, have used TVA flood data to plan new school construction. Shelbyville and Knoxville, Tennessee have used similar information in planning urban renewal projects.

The Federal Housing Administration, the Veterans Administration, the Public Housing Administration, and the Urban Renewal Administration have all used flood reports in processing home loans and in planning public projects. The departments of the state governments making use of the flood information include those dealing with highways and industrial development.

While this procedure for coping with flood problems will have its principal effect on the future growth of our communities, it has a smaller but significant effect on areas of cities which already have developed within flood danger zones. Our urban areas are not static. Residential areas turn commercial and business areas are converted to manufacturing. Urban renewal involves the razing of slums and the elimination of blighted areas. Highways, expressways, and belt parkways require remodeling much of the face of the modern city. Many of these changes may in one way or another involve habitation on the flood plain, and the flood potential must be considered in order to take the fullest advantage of the modernizing programs.

In recommending to the President and Congress a national program of flood damage avoidance, TVA has not sought to imply that present emphasis on corrective flood control measures should be discontinued or even diminished. There are hundreds of areas already intensively developed where

protective works are economically feasible. And there will always be industries and installations so intimately associated with the river that physical nearness is essential. National as well as local interest requires our awareness of this fact. Thus the T.V.A. stated in its report, "federal planning and development of projects which are parts of major stream regulation systems are essential. These systems, as under present programs, should continue to be financed largely by federal funds."

But the T.V.A. is convinced that the weight of the Federal Government—and it might be added, the weight of the engineering professions—must be thrown into a national effort to bring about the realization of the great responsibilities which states and communities must undertake if they are to prevent the disasters of the future.

The local government must appraise the local flood problem and determine the best solution, preparing and executing the necessary plans to accomplish that purpose. Engineering skills are essential in this process. The local officials are the individuals who must establish flood-plain regulations in a manner which will receive public acceptance and at the same time guide buildings away from flood-hazard areas. Here again, the counsel of trained engineers is most important. These same officials must be prepared to withstand the pressures from special interests which seek to overthrow or make exceptions to the regulations. Further, they must create a continuing awareness of the flood problem on the part of the public, planners, developers, and builders.

This new emphasis on flood avoidance should take a positive form as well as the negative action of preventive regulations. Zoning, building standards, and subdivision regulation do only half the job. Positive community planning and guidance, with incentives provided to induce development away from the flood plain, are essential.

Correspondingly as a matter of national policy it is wise to stimulate widespread interest among the state and local officials by positive incentives as well as through restrictive action. For example, it is necessary and desirable that the federal financial contribution to flood-control projects should be contingent upon the requirement that the city or state initiate and prepare engineering plans, and that they undertake the controls and guidance which will guard against the unnecessary spread of buildings and other improvements on the flood plains. At the same time, there must be an active program of education to make sure the communities realize their growing hazard, and that they are aware of the opportunities for guided growth which are available through the hydrologic data which government agencies can furnish.

In his address to the 1957 Governors' Conference at Williamsburg, Virginia, President Eisenhower deplored in vivid language the trend toward centralization of responsibility in Washington.

"Like nature," he said, "people and their governments are intolerant of vacuums. Every State failure to meet a pressing public need has created the opportunity, developed the excuse and fed the temptation for the national government to poach on the States' preserves. Year by year, responding to transient popular demands, the Congress has increased Federal functions. So, slowly at first, but in recent times more and more rapidly, the pendulum of power has swung from our States towards the central government."

Here, in dealing with floods, is a chance to reverse the swing of the pendulum. Indeed, unless it is reversed—unless the states and their local political subdivisions take up the responsibility—the vacuum will be filled with

disaster. For here is a field in which the Federal Government can only assist. The success of this entire country in coping with its growing flood problem will depend on how effectively local communities, working at the grass roots, deal with their individual problems. The job can never be accomplished by one level of government working alone.

We have the tools of science and government, and we have the knowledge of how to use them. Now we need the understanding and cooperation of all levels of government to put them to work.

There is no better way of summarizing these views than by quoting from the letter to Congress through which TVA transmitted its report, A Program for Reducing the National Flood Damage Potential:

TVA believes that local communities have the responsibility to guide their growth so that their future development will be kept out of the path of flood waters. With the States and communities of the Tennessee Valley, TVA has developed a means of putting this proposition into action. This experimental program has operated successfully for six years. It incorporates the concept of flood damage avoidance as a partner of flood control just as forest fire prevention is an indispensable arm of forest fire control. It is saving lives and property in the area while diminishing the future demands on the Nation for flood relief and flood control expenditures. We believe the same results can be accomplished by adapting this experience to other areas throughout the United States. The pace of river control development in relation to the even greater rate of urban encroachment makes it urgent that this broader concept be made a part of our national flood control policy.

The first of these is the fact that the system is not self-sufficient. It requires a constant input of resources from the outside world, and this is a major source of its weakness.

Secondly, the system is not flexible. It is rigid and inflexible, and this makes it difficult to adapt to changing circumstances. This is another major source of its weakness.

Thirdly, the system is not efficient. It is wasteful and inefficient, and this makes it difficult to maintain. This is another major source of its weakness.

Fourthly, the system is not secure. It is vulnerable to attack and sabotage, and this makes it difficult to trust. This is another major source of its weakness.

Fifthly, the system is not sustainable. It is not designed to last, and this makes it difficult to maintain. This is another major source of its weakness.

Sixthly, the system is not transparent. It is opaque and secretive, and this makes it difficult to understand. This is another major source of its weakness.

Seventhly, the system is not accountable. It is not responsible for its actions, and this makes it difficult to hold it to account. This is another major source of its weakness.

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THE FOURTH ROOT n - f DIAGRAM

By T. Blench,¹ F.ASCE

SYNOPSIS

A friction-factor design diagram, the "fourth root n - f diagram," is presented as an alternative to the Moody Diagram for engineers who prefer to use the Manning equation for the boundary condition called "rough." Essentially, this design diagram is a Moody Diagram in which the logarithmic basic phase lines have been linearized, the set of transition curves for one type of boundary material has been replaced by a few samples for several types, the set of linear "rough boundary" lines has been collapsed into one, and a special line has been added to relate Manning's n to roughness height. The scale of roughness height is not that of Moody; the formula used for the diagram deviates slightly from Manning's if compared over a large range of diameters, and deviates more from the logarithmic formulas. The diagram can be reproduced very easily to large scale from the equations given in the text and by scaling up the few transition curves, whose accuracy is not very important.

Note.—Discussion open until June 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 1, January, 1960.

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The construction and use of the diagrams are presented so that it can be reproduced for practical design and judged by results. Theory is unimportant for the purpose of the paper so it is given, in a condensed manner in the section, Principal Theoretical Points.

INTRODUCTION

Many engineers prefer to design for steady turbulent flow in uniform pipes and canals by using; (a) the Manning equation for the phase of flow associated with so-called "rough boundary" and, (b) other, apparently unrelated, formulas for transition and smooth boundaries. This procedure leaves them in doubt as to when they should make a change from one formula to another. On the other hand, textbooks are given increasing prominence to design by means of a friction-factor diagram fitted with design lines based on certain theoretical formulas. This diagram, the Moody Diagram² (Fig. 1), purports to cover all the phases of flow associated with rigid boundary roughnesses that can be expressed in terms of a single "roughness height," provided the fluid is Newtonian and free from solid suspension, and provided the pipe shape is circular. Means can be suggested for applying or adapting to other types of roughness, non-circular shapes, channels, non-cohesive boundaries, and fluids that carry suspensions, or are non-Newtonian; these means do not seem to be included in the original derivation.

Presumably there must be some good reasons for refraining from use of the Moody Diagram's practical advantages, and the author lists some probable ones in Appendix I. To overcome them constructively, the prime purpose of this paper has been made the presentation of a friction-factor design chart intended to give the practical advantages of that kind of plot, combined with the freedom to use Manning's n .

As theoretical speculations are out of place in the presentation of a design chart, for practical use the author proceeds, in the next two sections, to description and use and relegates abbreviated theory at the end, where the purely practical reader can ignore it without loss. The reader is assumed to have studied a first course in hydraulics and to have been acquainted with the manipulation of the Moody Diagram, as detailed in some texts.² However, the following exceedingly brief resume of the status of flow formulas may bring out the essential simplicity of the topic regarded as a whole.

Every flow formula with an alleged "theoretical" background is based, like the old-fashioned Chezy formula, $V = C \sqrt{R S}$, on an idealizing assumption, and lacks full logical justification; no formula of any origin has been supported uncontroversially by the whole range of practical data. The reasons for this include; the inability to describe turbulent flow, and the inability to measure natural roughness heights directly. All analyses of data to test theoretical formulas, or to produce formulas free from speculative bias, may be considered as attempts to relate Chezy's C to the physical factors upon which it depends, and Chezy's formula may be thought of as the fundamental one of the

² "Fluid Mechanics," by V. L. Streeter, McGraw-Hill, 1958, Chapter 4.

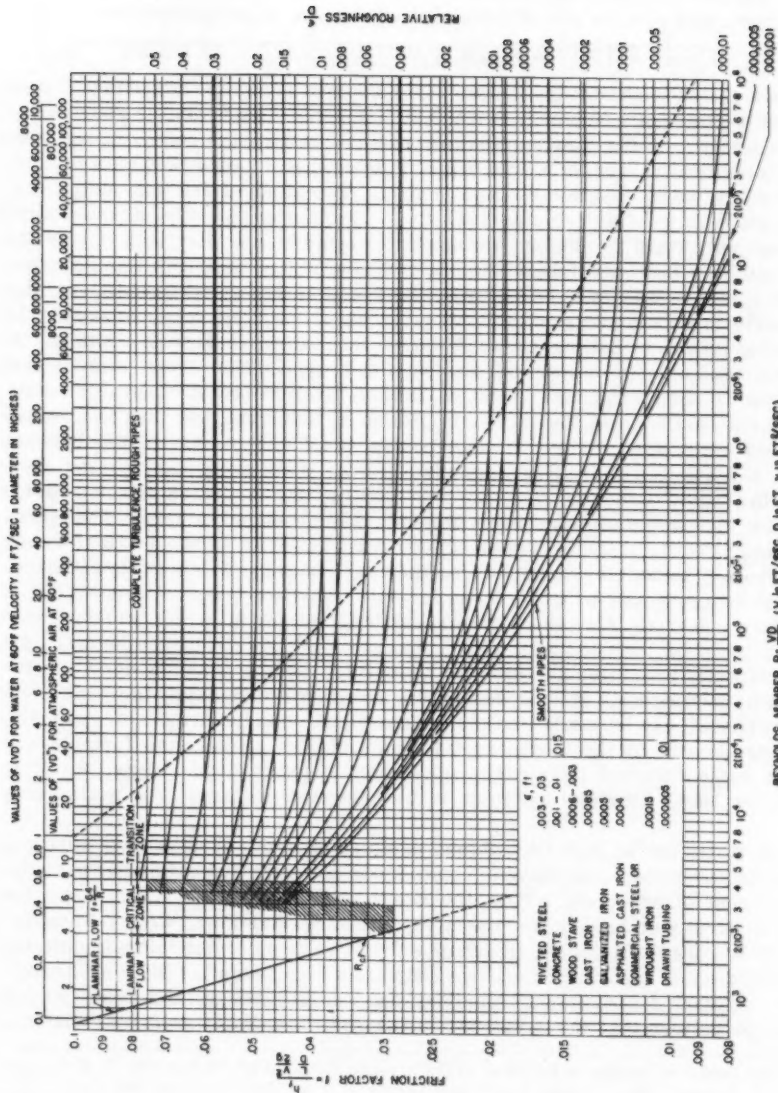


FIG. 1.—MOODY DIAGRAM

subject. However, the relationship among physical quantities of any kind can be reduced to a minimum number of terms by rearranging them into non-dimensional groups. This as now taught under the heading, "dimensional analysis." The method was well known at the end of last century, so Chezy's C was soon converted into its non-dimensional equivalent, f , as defined by:

$$C = \sqrt{8 g / f} \quad \text{equivalent to} \quad V = \sqrt{8 g r S / f} = \sqrt{2 g d S / f}$$

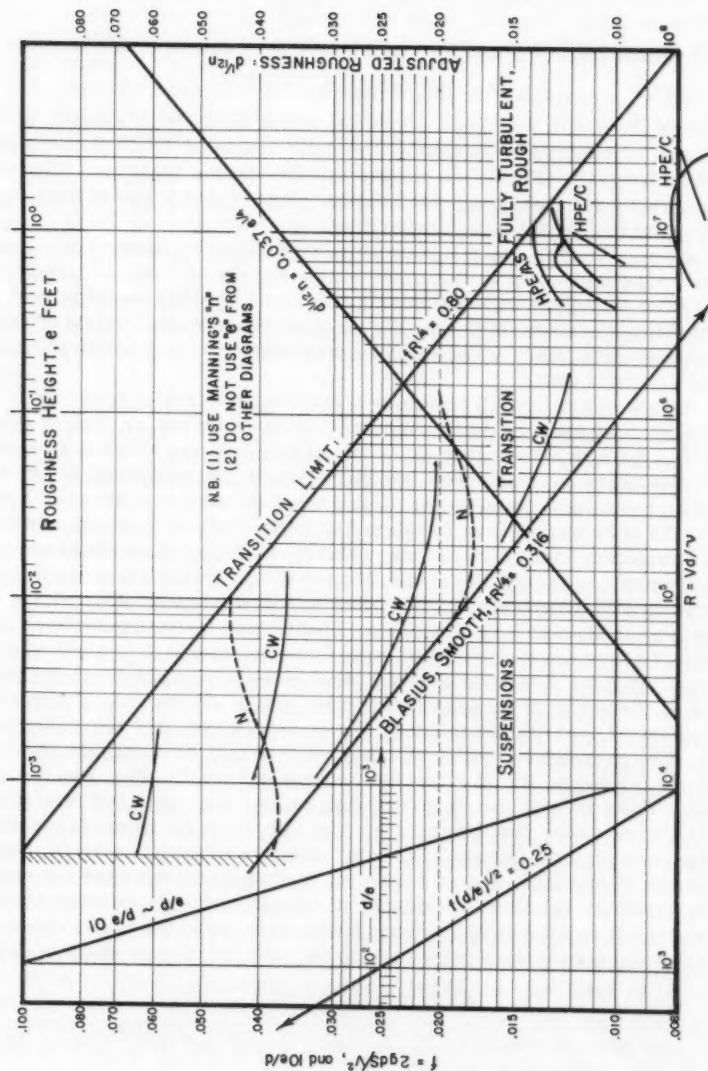
and now called the "friction-factor." Dimensional analysis showed that f must depend on Reynold's Number, $R = V d / \nu = 4 V r / \nu$, on "relative roughness," and on shape. So the obvious step towards discovering the behavior, and possibly functional nature, of f was to plot it against R for observed data for circular pipes (to eliminate the shape factor). This kind of friction-factor diagram plot was used as early as 1911, by T. E. Stanton^{3,4,5} who acknowledged dimensional ideas to Stokes, Helmholtz, Reynolds and Rayleigh during the nineteenth century; but did not attempt to fit the plotted data with lines of equal relative roughness, because natural roughness is not measureable. Stanton did perform experiments with pipes given equal relative roughnesses by the artificial means of cutting crossed screw-threads in them with depth of cut proportional to pipe diameter, and verified expectation from dimensional analysis for this condition. Plots of data on friction-factor diagrams established the existence of the two main phases of turbulent flow that is now associated with "smooth" and "rough boundaries," and the transition between them, and led to a search for formulas for the main ones. As various rival formulas came into use, the way was cleared for a design friction-factor diagram that would plot formulas of good repute, instead of data. The Moody Diagram, Fig. 1, is an example, based on curvilinear formulas (that do not plot straight on double-log paper) for the main phases and an ingenious mathematical compromise between them for the transition phase for one kind of boundary material. A table of "roughness heights" for different boundary materials must be included with such a diagram. These roughness heights, not being directly measureable, have to be defined by the formula used for the diagram; any formula of good repute will suffice provided that its "coefficient of ignorance" can be made linear by dimensional manipulation--for example, Manning, Bazin, or the fourth-root formula of Fig. 2--but each formula will have a scale different from every other and, being of good repute, will give about the same practical answers in use however different the scales may look.

It is worth noting that the subject is made difficult by attaching personal names to formulas, according to countries of use, putative origin, or particular interest of user, etc. Thus the same, or virtually the same, formulas carry a variety of names, for example, Manning-Strickler; Chezy-Fanning-D'Arcy-Weisbach. Actually, the whole subject reduces to one formula defining the coefficient of ignorance f and two ultimately true formulas (not yet agreed), one to express f for smooth boundary and one for rough; and there is good reason to believe that these two will prove to be derivable from one that can

³ "Similarity of Motion in Relation to the Surface Friction of Fluids," by T. E. Stanton and J. R. Pannell, Phil. Trans. Royal Soc. Series A, 1914, Vol. 214, pp. 199 - 224.

⁴ "The Mechanical Viscosity of Fluids," by T. E. Stanton, Phil. Trans. Royal Soc. Series A, Vol. 85, pp. 366 - 376.

⁵ "Friction, Section in Dictionary of Applied Physics," by Sir Richard Glazebrook, Mac-Millan & Co., London, 1922, pp. 356 - 363.

FIG. 2.—FOURTH ROOT n - f DIAGRAM

be divided into the two by proper expression of an equivalent roughness height and will give rise to a third for noncohesive boundary.^{3,4}

DESCRIPTION OF FOURTH ROOT n - f DIAGRAM

General Structure.—The term "fourth root n - f " refers to the equation

$$V = 2.0 (d/x)^{1/4} \sqrt{2 g d S} \dots\dots\dots (1)$$

which, with the fourth root replaced by the sixth, would be of exactly the functional form of Manning's equation. The main scales of f and R make it, like the Moody Diagram (Fig. 1), a classic friction-factor diagram. The scales are logarithmic for maximum convenience. Figs. 1 and 2 should be compared point by point for the following description:

The design line for smooth boundary is obtained by placing x = laminar film thickness and using a thickness value borrowed from an argument of Prandtl's for a slightly different case⁶; this gives the Blasius equation.⁷ The corresponding curve in Fig. 1 is based on a logarithmic formula which the author believes to lack a completely logical foundation and verification in the range of practical data.⁸

The design line for rough boundary is obtained by placing x = e , the roughness height, and appears in the bottom left-hand corner of Fig. 2 labelled $f(d/e)^{1/2} = 0.25$, which is just Eq. 1 in different form; above it is a line to convert d/e to e/d . This rough boundary line is the equivalent of the set of nearly horizontal e/d lines in Fig. 1 and could be used to draw such a set on Fig. 2 if the user wished (see Problem 3 below). Note e is not Moody's.

The Transition Limit line has its counterpart in Fig. 1 and is based on the same argument, so is parallel to the Blasius line; its location suits that of its counterpart, with a very slight adjustment to suit some field data.

Between the Smooth and Transition Limit lines is the zone where conditions cannot be represented by any simple formula. In Fig. 1 is a set of curves calculated from an empirical formula devised to be asymptotic to the Smooth line and to the set of horizontal Rough ones and to fit the data of Colebrook⁹ for commercial iron pipes; the data cover only about half the range of the curves. The curves are totally unlike ones that can be found for other materials. Accordingly, Fig. 2 gives four examples from the Fig. 1 curves, cutting them down to suit the observational range, and adds two examples of sand-roughness from the laboratory, and six of very large concrete and asphalted-steel tunnel linings. These examples are intended as a guide, so that the user can interpolate, and as a warning that whatever the user interpolates must be based on reliable experience; of course, there is no need to interpolate curves if an empirical formula, known to apply to the case is available.

To allow the appropriate value of e to be used the upward sloping line:

$$d^{1/12} n = 0.037 e^{1/4}$$

⁶ "Applied Hydro- and Aero-Mechanics," by L. Prandtl, and O. G. Tietjens, McGraw Hill, 1934, Dover Publications, 1957, Sec. 39.

⁷ "Fluid Mechanics," by V. L. Streeter, McGraw-Hill, 1958, p. 181.

⁸ "Unification of Flow Formulas," by T. Blench, Transactions Seventh Gen. Meeting Internatl. Assoc. for Hydr. Research. Lisbon, 1957. (Plus discussion)

⁹ "Turbulent Flow in Pipes, with Particular Reference to the Transition Region Between the Smooth and Rough Pipe Laws," by C. F. Colebrook, Proc. Institution of Civ. Engrs., 1959, Paper 5204.

is provided; its scales are to the right of and on top of the diagram (Fig. 2). It is most important to note that, in this formula, n is a value that is known to be applicable because it was found from observations of pipes of diameter equal to or not much different from d . The reason for this qualification is that Manning's n is believed to be slightly dependent on size of pipe, so that it has to be multiplied by $d^{1/12}$ to produce an "absolute roughness" that will not depend on size.

Construction of Diagram.—A practical value of the diagram is that it can be drawn rapidly to large scale on a couple of standard 2×3 cycle logarithmic papers joined together. Details for construction are:

i. The n , e line

The equation of this is $d^{1/12} n = 0.037 e^{1/4}$. Obviously it is the straight line joining $(e, d^{1/12} n) = (10^0, 0.037)$ to $(10^{-4}, 0.0037)$. Note that e defined here is not the e of the Moody diagram and must not be interchanged with it, even approximately.

ii. The smooth-pipe line

This is the well-known Blasius line, equation $f R^{1/4} = 0.316$, so can be drawn by joining $f = 0.0316$ at $R = 10^4$ to $f = 0.00316$ at $R = 10^8$ (a little outside the diagram).

iii. The transition limit line

This has the equation $f R^{1/4} = 0.8$, so is parallel to the Blasius line, whose slope is minus $1/4$, and passes through $f = 0.008$ at $R = 10^8$, that is, through the bottom right-hand corner of the diagram. It has been drawn as a compromise between the showing of the various transition curves (v. below) and the curved transition limit of the Moody Diagram.

iv. The fourth root line

The equation is $f (d/e)^{1/2} = 0.25$, so the line passes through $f = 0.0025$ at $d/e = 10^4$ (outside the diagram) and $f = 0.025$ at $d/e = 10^2$. (Note that the axis of d/e is marked half-way up the diagram.) This line is continued slightly to the left of the diagram and given an arrow end. The reason is to remind the user that he is approaching the condition where actual roughness height may be so large a fraction of the pipe diameter that the solution of a problem using diameter at the base of roughnesses may be much different than when using diameter at their tips. The formal e value should not be employed alone to decide this matter since, although it may give the maximum height of roughnesses in jagged rock tunnels fairly well, will tend to give roughness heights considerably larger than the apparent textural ones in relatively smooth materials like concrete. The $10 e/d$ against d/e line is for those who prefer to use e/d rather than d/e .

v. Curves in the transition zone.

These curves replace the Colebrook-White ones which cover the whole of the Moody Diagram's depth. Their object is to give the user an idea of the types of transition to expect for different materials, and a knowledge of the ranges of variable covered by well-known or authoritative field observations and laboratory experiments. They are widely spaced because the practical user is expected to sketch transitions in terms of (a) the given ones, (b) all other available information relevant to the specific problem, and (c) personal judgement. Their details are:

a. Marked C.W. (for Colebrook-White)

These are sketched roughly from a Moody diagram after having been checked against the original Colebrook-White paper,⁹ and seem to cover

the range of actual observation both horizontally and vertically. The Moody curves are based on a formula that fits the original data quite well but which has been extrapolated to cover part of the diagram where the foundations of the formula are of doubtful applicability. They apply to "new commercial iron pipes."

b. Marked N. (for Nikuradse)

These cover the Nikuradse artificially roughened pipe experiments² from the second roughest to the smoothest (the roughest being omitted as its correct diameter seems doubtful to the author). They probably represent concrete surface roughness free from form-marks, pipe joints, etc. Of course the e/d values in the diagram should not be compared with Nikuradse ones since they are devised to a different scale.

c. Marked HPE/C (for Hickox, Peterka, Elder/Concrete)

These are for large concrete tunnels, taken from Fig. 6 of Friction Measurements in the Apalachia Tunnel, by Rex A. Elder, Trans. ASCE. Vol. 123, 1958 and Fig. 19 of Friction Coefficients in a Large Tunnel, by G. H. Hickox, A. J. Peterka and R. A. Elder, Trans. ASCE Vol. 113, 1948; Fig. 19 relates to discussion by J. N. Bradley and S. P. Wing. It is to be noted particularly that they show the kind of trend to be expected from the N curves, but to a different degree, and that they are not easily reconcilable with the smooth pipe line of the Moody Diagram.

d. Marked HPE/AS (for Asphalted Steel)

This is from the Fig. 6 quoted above, for 1944 data, and concerns steel-lined large tunnel with a type of asphalt coating described in the reference.

vi. Zones

Below the smooth-pipe line, in the zone marked SUSPENSIONS, Fig. 2, no actual data for clean liquids should fall except due to errors of measurement. However, data¹⁰ for suspensions such as occur in sediment-bearing streams, paper pulp transport, etc., may fall below it. Above the transition limit, in the zone marked FULLY TURBULENT, ROUGH, data obey equations of the Manning type and the fourth root line is used. Between the smooth pipe line and the transition limit, in the zone marked TRANSITION, are points that can be defined only by the type of transition curve that happens to suit the particular boundary.

USE OF FOURTH ROOT n - f DIAGRAM

Effectively the diagram is used according to the rules that apply to a Moody Diagram,² so it needs no lengthy explanation. The following simple problems do not need "trial and error" and are framed mainly to give the reader minimum trouble in verifying that the diagram can be used correctly. The problems also illustrate some important practical points.

Problem 1.—Observations show that Manning's n is 0.013 for a lined tunnel of 20 ft. diameter. What are e , e/d , d/e , and f for "rough boundary"?

Answer.— $d^{1/12}n = 1.283 \times 0.013 = 0.0167$. Top line of diagram shows $e = 0.042$ ft. (Notice that the equation used in defining e makes it somewhat larger than would be imagined for concrete-like materials; it is not intended to give exact visual representation.) Then $d/e = 20/0.042 = 476$, and the fourth-root line shows $f = 0.0115$; the 10 e/d line above it shows $e/d = 0.0021$.

¹⁰ "Regime Behaviour of Canals and Rivers," by T. Blench, Butterworths Scientific Publications, Toronto, Canada, 1957, (Chapter 5).

Problem 2.—Continuing Problem 1, assume a 5-ft diameter concrete pipe-line, carrying flow in the "rough boundary" stage, has a boundary identical in nature with that of the 20-ft conduit. Assume further that the definition of e by the fourth-root diagram is physically perfect. What would be the value of Manning's n for the 5-ft pipe line?

Answer.—As $d^{1/12}$ n would have to be the same for both diameters $n_5/n_{20} = (20/5)^{1/12} = 1.122$. So $n_5 = 1.122 \times 0.013 = 0.0146$. This illustrates a feature of Manning's formula that is well known in canal practice,¹⁰ namely, that Manning's n , for exact design, has to be reduced for large channels that have apparently the same true boundary roughness as small ones.

Problem 3.—Experiments with a certain pipe in rough boundary stage showed that it had an $f = 0.02$. Assume it were tested with $R = 10^5$. What would be the value of f during the test, if the pipe were of (a) new commercial iron, or (b) concrete?

Answer.—This is solved exactly as with a Moody Diagram, but the Moody Diagram would compel the user to accept a Colebrook-White curve for commercial iron, and would have one ready drawn for him close to $f = 0.02$; this curve would run out to a horizontal tangent very close to the "transition limit." The fourth root n - f diagram would not usually have a curve near enough to be immediately useful. So the user would have to sketch a curve using the CW, N, etc. types of curve of the diagram as a guide, and supplementing them with practical experience; or may prefer to use the diagram as nothing more than a warning to use some transition formula, for example, a Hazen-Williams formula, known to suit the case. The curve to be sketched would have to be tangent to $f = 0.02$ at the transition limit line. For convenience, this particular problem has been posed so that an N and a CW curve will be available. They show that, if applied to the particular iron or concrete, the f -value for the former would be 0.0225 and of the latter 0.0185 approx.

Problem 4.—What are the lowest possible values of f at a $R = 10^7$ according to (a) Moody, and (b) n - f diagrams?

Answer.—0.008 and 0.0056, extrapolating the Blasius line outside Fig. 2.

Problem 5.—On an n - f diagram plot all the HPE concrete curves of Fig. 19 of the reference v.(c), (above), that may appear to be of N type if they were complete. Plot also the smooth-pipe line of the Moody diagram. Consider how many cross, or head across, that smooth-pipe line, how many could be converted to reasonable looking N curves tangent to it, and how many could be converted to reasonable looking N curves tangent to the Blasius line of the n - f diagram.

Answer.—Reader's judgement. The problem draws attention to the difficulty of obtaining data for a definite judgement on flow formulas, the doubtfulness of the Moody smooth boundary line at high Reynold's Numbers, and the danger of extrapolating theoretical formulas.

Problem 6.—How would you use the diagram to find velocities in channels?

Answer.—Usual practice, effective though not perfectly logical, is to replace any given channel of hydraulic radius r by a circular pipe of diameter $d = 4 r$ and find V for this pipe.

PRINCIPAL THEORETICAL POINTS

Derivation of Equations Relating n to f and e .—The definition of f may be written as

$$V = \sqrt{\frac{2 g d S}{f}} = \sqrt{\frac{8 g r S}{f}} \dots\dots\dots (2)$$

Write Manning's equation in the alternative forms:

$$\begin{aligned} V &= 1.49 r^{2/3} S^{1/2} / n \\ &= \frac{1.49}{4^{1/6} \times 8^{1/2}} \left(\frac{d}{g^3 n^6} \right)^{1/6} \sqrt{2 g d S} \\ &= A (d/e)^{1/6} \sqrt{2 g d S} \dots\dots\dots (3) \end{aligned}$$

where A is a constant that can be adjusted to fix the scale of e, and d is 4 r. Note that $\sqrt{g} n$ has the dimensions of (length)^{1/6} because Manning, like Chezy, saw no point in showing \sqrt{g} explicitly in his formula. Had he introduced \sqrt{g} he would have an improved n with the dimension (length)^{1/6}. Comparison with Eq. 2 shows that the Manning formula is for rough-boundary type and defines a roughness height e according to

$$f(d/e)^{1/3} = B \dots\dots\dots (4)$$

Canal experience shows that Manning's n is a function of d, but a comparatively large range of d is needed to demonstrate the effect. The author believes engineers who deal with large conduits have observed the same effect. The matter can be adjusted for canals by altering the index of r from 2/3 to 3/4, which would make V vary as (d/e)^{1/4} and indicate that Manning's should be multiplied by a proportional to r^{1/12} to convert it into an "absolute roughness" that would not depend on channel size. The postulate of a universal flow formula for all boundary types makes the 3/4 index seem most consistent. Accordingly an e scale has been defined by

$$f(d/e)^{1/4} = 0.25 \dots\dots\dots (5)$$

for constructing the diagram. Eliminating f between this and Manning's equation, via Eq. 2, gives:

$$d^{1/2} n = 0.037 e^{1/4} \dots\dots\dots (6)$$

also used in the diagram.

Consistence Between e and Visual Appearance of Roughness.—The author would be content to treat e as a code number to be attached to roughness types that would be recognized intuitively as an observer recognizes faces; this is successful with n. In fact, there would be convenience in arranging the constant in Eq. 5 so as to keep all e values between 1 and 100. However, in deference to Moody diagram values, and to the preference of a proportion of engineers for values that suggest visual roughness heights, the author has adopted the value 0.25 for the constant in Eq. 5. He believes this will give e-values that suggest something like the overall height of irregularities in unlined rock tunnels, which are the only practical roughnesses that are fairly common and are really assessable by eye. Obviously, as an e-value means nothing more than the height of some standard roughness that gives the same resistance as another roughness that may be of quite different appearance, boundaries of comparatively smooth texture that probably owe much of their resistance to nontextural effects like form marks, irregularity of section alignment, etc., will not be too well represented by this particular e scale; in fact, concrete boundaries will have rather higher values of e than might be expected.

Inconsistence of Manning Formula and Moody Diagram.—To compare Moody and Manning rapidly let $B = 0.02$ in Eq. 4. Call the e defined by this equation e' and reserve e for the Moody Diagram. Then the equation gives $f = 0.02$ for $d/e' = 1000$, and the Moody Diagram gives f almost the same for $d/e = 1000$. So, for any one pipe, $e = e'$ at this point. Now consider a pipe of 1 ft diameter with $e = e' = 0.001$ ft. Suppose a 20-ft diameter conduit is to be made of exactly the same material. Then the believer in Manning would believe that $d/e' = 20,000$, and the believer in moody would believe that $d/e = 20,000$. Eq. 4 would predict $f = 0.0074$ for the former and the Moody Diagram would predict $f = 0.0107$ for the latter; this deviation is more than a practising engineer would accept. If it is assumed, for argument, the Manning estimate $f = 0.0074$ is correct, then the Moody diagram shows that $e/d = 0.000006$, or $e = 0.00012$, which is about one-eighth of the e that applied to the identical 1-ft pipe. This raises the question of whether an observer could fail to notice a change in boundary appearance when e was shrunk to one-eighth; if not the inconsistency of Manning and Moody is clear. If the author's belief, already discussed, that Manning requires the deliberate reduction of n for large pipes, is correct, then the Moody diagram differs from Manning in the same direction that Manning differs from the true.

Universal Flow Formula.—Belief in the existence of a universal velocity distribution (not in its particular equation) implies belief in a universal flow formula. Accepting the latter belief consider the formula:

$$V \propto (d/x)^{1/4} \sqrt{2 g d S}$$

Then, as discussed elsewhere^{8,10}:

i. If x = laminar film thickness δ , and the argument used by Prandtl is applied to deduce that the relative thickness of a boundary layer is inversely as the Reynold's Number, we find that d/x is replaced by the square root of $V d/\nu$, and the formula becomes that of Blasius.

ii. If $x = (\nu F_s)^{1/2}/F_b$ where F_s and F_b are the side and bed factors for a canal of cohesive erodible sides and sand bed, moving with small change, in dune formation, the formula turns into the flow formula for canals of regime type; but d now measures depth.

iii. If $x = e$, for rough rigid boundary, then the formula is the Manning one with the index of d changed from $2/3$ to $3/4$.

It is to be noted that case iii cannot be checked as well as the others from data because one cannot obtain pipes or channels of different size with boundaries that can be guaranteed identical. Case ii has very wide verification, and the boundary, being formed from transported load, is not so indefinite as in the rigid boundary case. Case i should be particularly reliable as the roughness depends entirely on the fluid viscosity. Comparison of the Nikuradse smooth pipe experiments, practically the same as Stanton's, with the Colebrook-White ones, shows that they do not prove that the Blasius formula breaks down at high Reynold's Numbers; all that is shown is that, no matter how smooth a pipe looks, it starts to become rough at some high Reynold's Numbers. The data plotted in the transition zone at the bottom of the n - f diagram also support the Blasius line at very high Reynold's Numbers.

NOMENCLATURE

A, B =	dimensionless constants;
C =	Chezy's co-efficient;
d =	diameter of pipe;
e =	roughness height;
f =	friction factor;
g =	acceleration of gravity;
n =	Manning's n;
r =	hydraulic radius;
R =	Reynold's number;
S =	loss of head per unit length, hydraulic gradient;
V =	mean velocity over section in turbulent flow; and
ν =	Kinematic viscosity

APPENDIX 1.—PROBABLE OBJECTIONS TO USE OF MOODY DIAGRAM

In the author's opinion engineering neglect to use the Moody diagram does not extend to deprecating the insight it gives into the nature of friction factor and probably rests on objections such as:

i. The Manning formula is reasonably reliable for large conduit work in or near the rough boundary stage; the Moody diagram is relatively unsatisfactory.

ii. The "smooth pipe" line of the Moody diagram is somewhat inconsistent with large conduit data.

iii. The curves of the Moody diagram have very complex formulas.

iv. The Colebrook-White transition curves on the diagram which have been extrapolated well beyond their data, apply only to new commercial iron pipes, and are quite inapplicable to concrete.

v. The experimental data of roughness height were not perfect but the relative roughnesses of the diagram have been extrapolated to several hundred times the observed values.

vi. Texts on fluid mechanics have made the tactical mistake of discussing mathematically obscured, but patently illogical, "proofs" of the logarithmic flow formulas upon which the diagram is based. Actually, the formulas rest on nothing more than the fact that a logarithmic velocity distribution is empirically a good fit to a circular pipe; Prandtl produced a proof for it under conditions which he was careful to explain did not apply to a circular pipe at all.

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GENERALIZED DISTRIBUTION NETWORK HEAD LOSS CHARACTERISTICS

by M. B. McPherson,¹ M. ASCE

SYNOPSIS

The analysis of complex distribution systems can be expedited by means of principles developed in this paper. Head losses over a wide range of demand and equalizing storage rates can be calculated directly, based on only two or three complete network analyses, under design assumptions employed in normal practice.

INTRODUCTION

There are presently two principal devices in use for the detailed calculation of head losses in complex distribution networks. Calculations can be performed through direct analogy with a McIlroy Network Analyzer^(1-a)² or by successive iteration via a digital computer utilizing a Hardy Cross relaxation technique.^(2-a,2-c) Regardless of the machine employed, computations for a large number of system demands and supply combinations for a given network are time-consuming and expensive. A new method of calculation requires only a minimum of network analyses through the use of generalized system head loss characteristics recently deduced. The method is consistent with the restrictions imposed by commonly used design assumptions.

Note.—Discussion open until June 1, 1960. Separate Discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 1, January, 1960.

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² References in numerals, (thus 1-a) refers to corresponding items in bibliography.

PROPORTIONAL LOADS

In preparing for a distribution system analysis the network is reduced or "skeletonized" to a principal, or arterial, network. Time and cost considerations preclude the determination of pipe resistance coefficients by field measurement for other than the primary feeders comprising the arterial system. Further, the inclusion of pipes of small carrying capacity would become an unnecessary burden in the analyses and would neither reduce the influence of necessary assumptions nor improve the overall accuracy of the results.

Existing demands (loads) must be somewhat arbitrarily apportioned and consolidated and then concentrated at points on the arterial network. Average consumption rates for commercial and industrial users can nearly always be obtained from the more extensive readings taken on large meters. Appraisal of annual average domestic consumption from meter readings is not always practicable and recourse must often be made to sector distributions (namely, waste-and-leakage surveys).

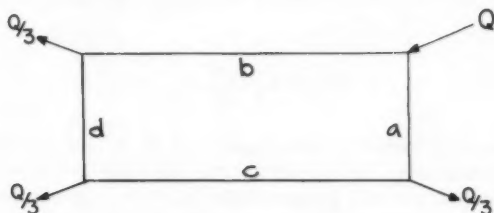
An hourly record of total demand variations for a system, or separate district, is normally available. Unfortunately, the time-variation of individual or even groups of local loads is seldom known. By default it is assumed that each local consolidated load fluctuates in direct proportion with the total system demand. The term "proportional load" will describe the typical design assumption that each individual consolidated load varies or fluctuates about its mean in direct proportion to the total system (or district) fluctuations. Thus for example the maximum and minimum hour demands of the average and maximum day are taken as fixed multiples not only of the annual averages for the district as a whole but also for each individual local load. Skeletonizing, estimating and consolidating of loads, and assuming complete proportionality for each load are justified on two bases: (1) design is usually directed towards satisfying projected future demands under estimated or planned future conditions, and (2) these same estimations and assumptions lead to calculated system head losses for a class of existing networks which usually agree remarkably well with field system head loss measurements. It must be recognized that while the assumption of proportional loads is suited to applications for predominantly residential systems or districts, a satisfactory correlation between calculations and field measurements is less likely to be obtained where there is a heavy concentration of industrial and/or commercial loads.

PROPORTIONAL LOAD HYDRAULICS

In comparing two or more total system demands in a proportionally loaded network, a head loss balance will be achieved only when the flow of each pipe is a fixed proportion of each of the individual total demands. An elementary, specific, single-loop example of this relationship is offered in Fig. 1. The total sendout is divided equally among the three loads in the example. Flow coefficients have been assigned arbitrarily to each of the four pipes. For a balance in head loss the flow in each pipe is a fixed part of the sendout, Q , for the given loads and pipe coefficients. Head loss has been taken as being a function of the square of pipe flow rate, but the use of any other power of m (see Notation) would also result in a fixed relationship and the distribution of numerical ratios would be different.

In Fig. 2, Case I, is shown a balanced, simple, two-loop network given in

reference 2-a, with $m = 1.85$. The sendout, Q_p , and each of the five loads have been doubled in Case II. Also, the flow in each pipe has been doubled with the corresponding loss in each pipe then becoming 21.85 times that for Case I, and Case II is in balance. It may be seen that the head losses for proportional



GIVEN:

$$h_a = 7KQ_a^2$$

$$h_b = 6KQ_b^2$$

$$h_c = 8KQ_c^2$$

$$h_d = 9KQ_d^2$$

EQUATIONS TO BE SOLVED SIMULTANEOUSLY:

$$Q_a + Q_b = Q$$

$$Q_c = Q_a - Q/3$$

$$Q_d = Q_b - Q/3$$

$$Q_c + Q_d = Q/3$$

$$h_a + h_c = h_b + h_d$$

SOLUTION:

$$Q_a = (82/168)Q$$

$$Q_b = (86/168)Q$$

$$Q_c = (26/168)Q$$

$$Q_d = (30/168)Q$$

THEREFORE, WITH PROPORTIONAL LOADS FLOW IN EACH PIPE IS A FIXED RATIO OF THE TOTAL DEMAND, Q

FIG. 1.—EXAMPLE OF PROPORTIONAL LOAD HYDRAULICS

loads balance when the flows in each line are also proportional to the total demand. For example, in pipes e' and e, Cases I and II:

$$2^{1.85} (19.9 \text{ ft} + 4.2 \text{ ft} = 24.1 \text{ ft}, = 87.0 \text{ ft} = 71.8 \text{ ft} + 15.2 \text{ ft},$$

and in pipes b, d' and d, Cases I and II:

$$2^{1.85} (2.6 \text{ ft} + 14.8 \text{ ft} + 6.7 \text{ ft} = 24.1 \text{ ft}) = 87.0 \text{ ft} = 9.4 \text{ ft} + 53.4 \text{ ft} + 24.2 \text{ ft}.$$

If the overall loss between junctions e'-b and e-d is designated Σh_1 , it follows that

$$\Sigma h_1 = K_1 Q_d^m \dots \dots \dots (1)$$

where K_1 is a constant between both cases and m is 1.85 for all pipes in the example. This fundamental relationship will hold for the head loss between any two points in a network with proportional loads, but the numerical value of K_1 will be different for each pair of points selected. However, Eq. 1 is applicable only when Q_p equals Q_d . The value of m in Eq. 1 is the same as the exponent for all individual pipes; the equation cannot be applied where mixed m -values are used for different network components.

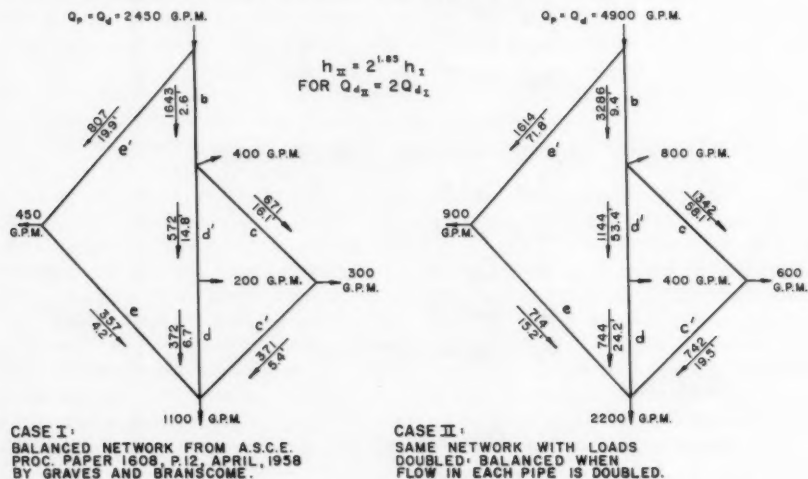


FIG. 2.—CHARACTERISTICS OF CONSTANT PERCENTAGE DEMANDS WITHOUT STORAGE

PROPORTIONAL LOAD CHARACTERISTICS WITHOUT EQUALIZING STORAGE

A more complicated network is shown in Fig. 3. This is the arterial system for the "existing" (1958) Belmont Gravity District in Philadelphia. As in Figs. 1 and 2, sendout is direct with no equalizing storage on the system. In Fig. 4 head losses are plotted from the filtered water basin to three network points (identified in Fig. 3) for four different total demand rates as calculated using the Philadelphia Water Department's McIlroy Network Analyzer. The solid lines through the calculated points are for $m = 1.85$, the flow exponent used for all pipes in the network. It may be seen that for a system without equalizing

storage the head loss between any two locations can be calculated directly from the results of one run, for a network with proportional loads, using Eq. 1.

Also plotted in Fig. 4 are corresponding field data. Considering the attainable accuracy of this type of data and the various assumptions and limitations of the analyses, correlation is good except for the "Southwest" station.

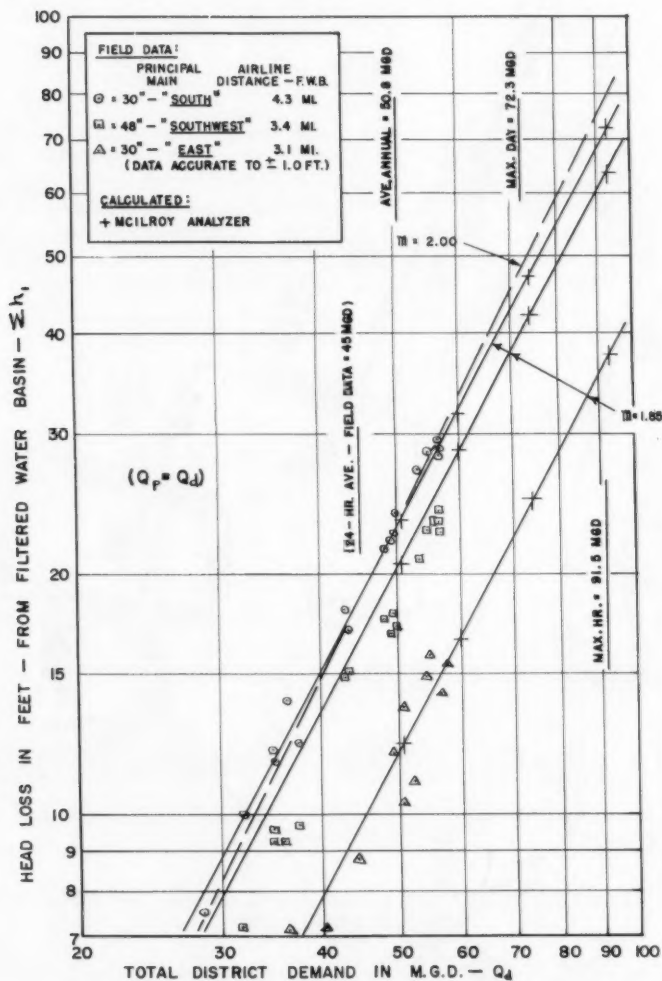


FIG. 4.—SYSTEM HEAD LOSSES, BELMONT GRAVITY DISTRICT, "EXISTING" NETWORK, SEE FIG. 3

It was found belatedly that the Hazen-Williams C of 69 near the station in question (see 4,200 ft of 48 in. in Fig. 3) should have been 75; correcting this error brought the line representing the analyses in Fig. 4 much closer to the

"Southwest" field data without noticeably affecting the other points. An error in a field C-value measurement was thus found through the analyses.

Part of the scatter of field data can be attributed to the range of accuracy to which the gages could be read, but the probability of inexact proportionality of local loads must be considered at least partially responsible.

Of the 50.8 mgd current average annual demand in this district, 17% is used by industries. The procedures employed in preparing the network for analysis, including the assumption of proportional loads, appear to be consistent with field measurement despite moderate industrial usage. Similar correlation has been obtained in high service districts with small industrial use.

The demand range represented by the field data in Fig. 4 is quite typical. Seldom is a rate as high as the current maximum hour of the maximum day obtained or closely approached. The projected future maximum hour rates used in design are generally greater than the current or recent peaks due to anticipated per capita consumption increases. The solid lines of Fig. 4 constitute a projection beyond the field data with $m = 1.85$. Various references have suggested an m of 2.00 for "old" pipes (the C-values in Fig. 3 are mostly below 100). A line of fit for $m = 2.00$ for the "South" gaging station is shown (dashed line) in Fig. 4. Much of the larger piping will have to be cleaned and lined to meet adequately future anticipated increased demands at satisfactory pressures. The analysis for future conditions thereby included a mixture of "new" and "old" pipes, for which an m of 1.85 was applied throughout. In network analyses an m of 1.85 is generally used for all pipes regardless of their condition, for reasons discussed elsewhere.⁽³⁾ The remaining examples will be for an m of 1.85.

Field engineers have observed that the head loss from a pumping station (or from gravity-feed storage) to a distant point in the distribution system approximates a function of the sendout rate to the 1.85-power. When the minimum allowable fire pressure is not reached in N.B.F.U. local grid field fire flow tests, the pressure residuals are scaled to the allowable using a 1.85-power relation. The general procedure for adjusting fire flow test results has been described by Hudson.⁽⁴⁾ It has been demonstrated here that a generalization of network losses can be achieved under the assumption of proportional loads. General agreement between analyses and field data taken during periods of normal consumption suggests that there is some validity to the design assumption of proportional loads. However, a fire load cannot be regarded as a representative proportional load. The heavy concentration of loading at a point obviously cannot be considered in the same category as the normal local consumption load. Therefore, the results of fire flow tests or analyses will usually depart considerably from those for proportional loading.

PROPORTIONAL LOAD CHARACTERISTICS WITH EQUALIZING STORAGE

Eq. 1 is restricted to a system without equalizing storage. Rearranging Equation (1):

$$\frac{\Sigma h_1}{Q_d^m} = K_1 = \text{constant},$$

and realizing that Eq. 1 represents a special limit where $Q_d = Q_p$, or Q_p/Q_d

= 1, it is reasonable to assume that the left side of the above rearranged equation might be a function only of Q_p/Q_d , or,

$$\frac{\Sigma h}{Q_d^m} = f(Q_p/Q_d).$$

Assuming the functional relationship to be satisfied by a coefficient and an exponent,

$$\frac{\Sigma h}{Q_d^m} = \phi(Q_p/Q_d)^n \dots \dots \dots (2)$$

note that for the limit $Q_p/Q_d = 1$, $\Sigma h = \Sigma h_1$ and ϕ become equal to K_1 . That

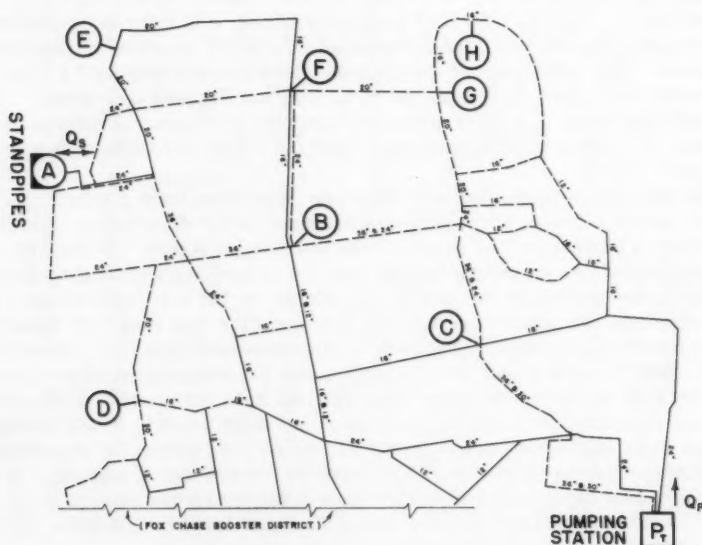


FIG. 5.—TORRESDALE HIGH SERVICE DISTRICT, ARTERIAL PIPING NETWORK FOR B-24-25R STUDY, NOT TO SCALE, AIRLINE DISTANCE FROM P_T TO A APPROXIMATELY 5.10 MILES (FOX CHASE BOOSTER DISTRICT ISOLATED)

the relationship given by Eq. 2 indeed satisfies both the case of $Q_p/Q_d = 1$ without equalizing storage and also a range of about 0.5 to 2.0 with equalizing storage will be demonstrated with $m = 1.85$. For flow to storage $Q_p/Q_d > 1$ and for flow from storage $Q_p/Q_d < 1$.

Once the head loss between two given points is known for two different magnitudes of Q_p/Q_d simultaneous solution using Eq. 2 will lead directly to ϕ and n from which the head loss for any other Q_d and Q_p/Q_d can be immediately calculated. The use of peak day maximum and minimum hour demands is

preferable since these span a large range of Q_p/Q_d and usually represent critical design conditions.

The revised arterial network for a study of Philadelphia's Torresdale H.S. (High Service) District is shown in Fig. 5. Corresponding Σh from the pumping station (P_T) to eight points including the equalizing storage site are given in Table 1. Runs Nos. 3 and 18 were original routine design analyses; the values from these two runs were used to calculate the ϕ and n for each location for comparison with the "measured" losses for eighteen special runs. The "calculated" head losses from the pumping station to the standpipes (Location A) are quite close to the "measured." Values for the seven points in the arterial network compare favorably. The eighteen special runs were performed with dispatch and the head loss differences would have been less had the meter adjustments on the Analyzer been made more precisely. For the

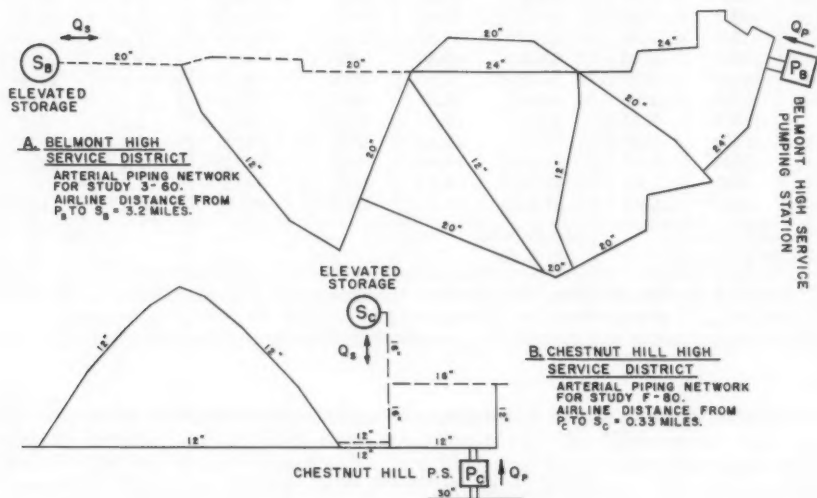


FIG. 6.—ARTERIAL PIPING NETWORKS

sake of computational clarity, inserting the test values for Location A of run No. 3 in Eq. 2:

$$\frac{36.1 \text{ ft}}{(63.0)^{1.85}} = 0.0169 = \phi (0.750)^n,$$

and of run No. 18:

$$\frac{59.5 \text{ ft}}{(28.4)^{1.85}} = 0.122 = \phi (1.67)^n.$$

Solving for n ,

$$n = \frac{\log (0.122/0.0169)}{\log (0.67/0.750)} = 2.46,$$

TABLE 1.—HEAD LOSS, Σh , IN FEET, FROM PUMPING

No.	Q_d , mgd	Q_p/Q_d	Location A		Location B		Location C	
			Meas.	Calc.	Meas.	Calc.	Meas.	Calc.
1	52.5	0.667	18.0	19.3	21.5	21.2	13.8	13.2
2	52.5	0.680	19.1	20.3	22.3	22.1	14.2	13.8
3	63.0 ^a	0.750	36.1	36.1	38.1	38.1	23.2	23.2
4	52.5	0.750	26.7	25.8	28.1	27.2	17.2	16.5
5	42.0 ^b	0.750	17.6	17.1	18.5	18.0	11.3	11.0
6	42.0	0.846	24.2	23.0	24.5	23.2	14.1	13.7
7	52.5	0.905	42.3	40.9	42.8	40.4	23.9	23.6
8	47.2	1.00	44.2	43.0	42.3	41.0	23.6	23.4
9	43.5	1.00	38.2	36.9	36.8	35.2	20.4	20.1
10	39.4	1.00	31.9	30.8	30.7	29.4	17.2	16.7
11	35.5	1.00	26.7	25.4	25.7	24.2	14.2	13.8
12	31.5	1.00	21.2	20.3	20.5	19.4	11.4	11.0
13	37.7	1.25	53.8	49.5	47.9	43.3	24.3	23.5
14	31.5	1.25	36.9	35.2	33.0	31.2	17.8	16.8
15	25.1	1.25	24.8	23.5	22.1	20.5	11.9	11.0
16	18.9	1.25	14.1	13.7	13.0	12.1	7.0	6.6
17	18.9	1.50	21.1	21.4	18.6	17.5	9.5	9.3
18	28.4 ^c	1.67	59.5	59.5	47.2	47.2	24.1	24.1
19	23.6	1.67	42.0	42.1	34.1	33.6	17.2	17.1
20	18.9 ^d	1.67	27.8	27.9	23.0	22.3	11.7	11.4
Value of ϕ			0.0344		0.0328		0.0187	
Value of n			2.46		2.11		1.89	

^a Run B-25R, Max. Hr. Max. Day Demand, "Calculated Σh ," ϕ and n based on "Meas-
"Calculated Σh ," ϕ and n based on "Measured Σh ." ^d Min. Hr. Ave. Day Demand.

and substituting n in either of the first two equations yields $\phi = 0.0344$, or

$$\frac{\Sigma h}{Q_d^{1.85}} = 0.0344 (Q_p/Q_d)^{2.46}.$$

This last equation was used to obtain the "calculated" Σh -values for Location A in Table 1. It must be realized that these values of ϕ and n would be changed if either the loads or the piping in the network of Fig. 5 were modified in any way, or if m was changed. These constants are particulars of the given network combination.

In determining head losses for use in a special study of balanced pump-network-storage conditions for the Belmont H.S. District,⁽⁵⁾ a total of nineteen runs had been made before the functional nature of Eq. 2 was fully appreciated. The arterial network for the balance study is given in Fig. 6A and represents expected interim conditions in the near future. The measured head losses for the nineteen runs are given in Table 2. The ϕ of 0.135 and n of 3.70 were determined from a log-plot of $\Sigma h/Q_d^{1.85}$ versus Q_p/Q_d . The "calculated" Σh are in good agreement with the average-fit to Eq. 2 despite the large range in magnitude of Σh .

In both the Torresdale H.S. and Belmont H.S. districts the equalizing storage location is on the side of the distribution system opposite from the pumping station. Elevated storage for the Chestnut Hill H.S. District is planned to

STATION, P_T , TO GIVEN LOCATION FOR FIG. 5 WITH $m = 1.85$

Location D		Location E		Location F		Location G		Location H		No.
Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	
23.0	22.4	25.2	25.2	27.7	27.2	34.7	32.4	47.0	43.3	1
23.9	23.4	26.3	26.3	28.3	28.3	35.0	33.5	47.0	44.5	2
40.5	40.5	45.4	45.4	48.0	48.0	55.7	55.7	72.0	72.0	3
29.5	29.0	32.4	32.4	34.1	34.2	40.1	39.7	51.9	51.3	4
19.2	19.1	20.9	21.4	21.9	22.6	26.2	26.3	34.0	34.0	5
24.4	24.8	27.4	27.6	27.9	28.6	31.1	32.4	38.9	40.5	6
43.5	43.4	47.0	48.1	48.0	49.3	52.2	55.0	63.5	67.5	7
44.1	44.2	46.8	48.8	46.9	49.1	50.0	53.8	59.1	64.1	8
38.5	37.9	40.5	40.9	40.4	42.2	42.4	46.2	50.0	55.1	9
31.9	31.6	33.7	34.9	33.5	35.2	35.4	38.5	41.6	45.9	10
26.8	26.1	28.1	28.8	28.0	29.0	29.1	31.8	34.0	37.9	11
21.5	20.9	22.6	23.1	22.3	23.2	23.2	25.4	27.0	30.3	12
52.3	47.3	54.1	51.6	49.9	50.0	50.0	52.4	54.7	58.6	13
35.9	33.7	37.0	36.9*	36.5	35.8	36.5	37.5	39.9	41.9	14
24.0	22.1	25.0	24.3	24.9	23.5	25.0	24.6	27.0	27.9	15
13.8	13.1	14.8	14.4	14.0	13.9	14.1	14.6	15.2	16.3	16
19.9	19.5	20.8	21.1	19.9	19.8	19.9	20.0	20.8	21.3	17
52.2	52.2	56.2	56.2	52.0	52.0	51.2	51.2	52.9	52.9	18
37.6	37.1	39.1	40.0	37.0	37.0	36.7	36.5	37.8	37.5	19
25.0	24.6	26.7	26.5	24.6	24.4	24.1	24.2	24.8	24.9	20
0.0354		0.0390		0.0393		0.0431		0.0514		
2.16		2.11		1.94		1.74		1.455		

ured Σh .^b Max. Hr. Ave. Day Demand. ^c Run B-24R, Min. Hr. Max. Day Demand.

be located in about the center of that system with the pumping station on the edge of the district, as shown schematically in Fig. 6B. (This is the smallest distribution district in Philadelphia and the only one in which fire flows governed the design of future arterial piping requirements.) The "measured" head losses between the pumping station and the elevated storage site for the four principal test runs from the final report for this district (written in 1956) are presented in Table 3, together with head losses "calculated" by using the data of the last two runs. The first two runs were for anticipated average day hourly extremes and the last two for maximum day hourly extremes. All four runs were carefully made. If the generalized characteristics given by Eq. 2 had been developed prior to these tests the first two runs could have been checked for consistency or merely calculated.

In Fig. 7 is shown the arterial network for one of the several studies performed some time ago for the Roxborough H.S. District. This district is more complex, having two pumping stations and two equalizing storage sites. The measured head losses for the Fig. 7 piping are given in Table 4. These data are offered to show that a more complex system can be generalized. The A and B series of runs in Table 4 are for average day demand extremes and the C and D series are for maximum day extremes. One of the loads in this study was not proportional; the Chestnut Hill P.S. was represented as a constant load for each of the two 24-hr average demands, but constituted only about 6% of the averages for the total Roxborough district. The "calculated" head losses from

the West Oak Lane P.S. to the three system points are quite close to the "measured" ones. To correlate the Roxborough P.S. head losses, greater consistency was achieved by algebraically combining P_R and S_R which are at almost the same position hydraulically. The "calculated" head losses for runs No. 6C and 6D are close to the "measured" ones; office notes taken at the time of the tests indicate that some of the "fluisters" representing pipes located between P_R and R-2, but nearer R-2, were found to be operating improperly during the

TABLE 2.—SYSTEM HEAD LOSS CHARACTERISTICS FOR FIG. 6A WITH $m = 1.85$

Q_d , mgd	Q_p/Q_d	Σh from P_B to S_B , feet	
		Measured	Calculated
19.9	0.698	9.0	9.0
12.3	0.757	4.0	5.0
11.2	0.760	3.4	4.3
12.3	0.797	5.8	6.1
10.3	0.827	4.8*	5.0
12.3	0.829	7.2	7.0
11.2	0.830	5.1	5.9
9.7	0.883	6.6	5.7
10.3	0.942	8.2	8.1
5.1	1.00	3.2	2.8
12.3	1.00	14.0	14.0
7.6	1.00	6.3	5.9
7.3	1.17	9.1	9.4
7.3	1.28	13.7	13.1
5.1	1.65	20.0	17.5
5.1	1.73	22.0	20.7
5.1	1.83	25.2	25.5
5.1	1.90	29.0	29.6
7.0	1.99	60.0	62.4

$$\phi = 0.135$$

$$n = 3.70$$

TABLE 3.—SYSTEM HEAD LOSS CHARACTERISTICS FOR FIG. 6B WITH $m = 1.85$

Q_d , mgd	Q_p/Q_d	Σh from P_C to S_C , feet	
		Measured	Calculated
0.89	1.69	6.2	6.0
2.04	0.735	1.8	2.1
1.17	2.05	18.4	18.4
3.48	0.690	4.6	4.6

$$\phi = 1.46$$

$$n = 3.12$$

No. 4 series and were replaced prior to the No. 5 and No. 6 series of runs. In retrospect it is therefore concluded that the "calculated" head losses for the No. 4 series are probably more accurate than the "measured." The data of Table 4 represent some extreme combinations: the No. 4 series is for equal sendout rates from the two pumping stations, the No. 5 series is for an outage of the Roxborough P.S. and the No. 6 series is for an outage of the West Oak Lane P.S. Had the procedures described here been known at the time these

TABLE 4.—SYSTEM HEAD LOSS CHARACTERISTICS FOR FIG. 7 WITH $m = 1.85$

Run No. (1)	Qd, mgd (2)	Roxborough Pumping Station, P _R										West Oak Lane Pumping Station, P _W					
		Σh , in feet, from P _R to given location										Σh , in feet, from P _W to given location					
		Q _{PR} /Q _d (3) ^a		R-1		R-2		R-3		Q _{PR} /Q _d (10)	R-1		R-2		R-3		
				Meas. (4)	Calc. (5)	Meas. (6)	Calc. (7)	Meas. (8)	Calc. (9)		Meas. (11)	Calc. (12)	Meas. (13)	Calc. (14)	Meas. (15)	Calc. (16)	
4A	26.9	0.550		18.4	18.6	18.3	11.4	12.0	8.9	0.351	9.0	7.0	8.9	7.2	2.6	5.5	
4B	9.2	0.326		1.4	0.8	4.1	0.4	3.9	0.3	1.03	7.0	6.9	10.0	10.4	9.5	8.9	
4C	17.4	0.540		7.6	7.9	8.1	4.9	5.3	3.8	0.379	3.9	3.6	4.5	4.1	1.7	2.9	
4D	7.9	0.392		1.6	0.9	2.5	4.8	2.3	0.4	0.835	3.4	3.6	4.3	5.0	4.2	4.3	
5A	26.9	—		—	—	—	—	—	—	0.703	25.0 ^b	25.0	33.8 ^b	33.8	27.0 ^b	27.0	
5B	9.2	—		—	—	—	—	—	—	2.05	24.6 ^b	24.6	43.9 ^b	43.9	43.0 ^b	43.0	
5C	17.4	—		—	—	—	—	—	—	0.758	13.3	12.8	17.2	17.7	14.6	14.3	
5D	7.9	—		—	—	—	—	—	—	1.67	12.5	12.7	22.0	21.6	21.0	20.3	
6A	26.9	0.900		55.8 ^b	55.8	42.7 ^b	42.7	38.5 ^b	38.5	—	—	—	—	—	—	—	
6B	9.2	1.35		19.0 ^b	19.0	17.4 ^b	17.4	17.7 ^b	17.7	—	—	—	—	—	—	—	
6C	17.4	0.920		26.4	26.2	20.9	20.2	19.0	18.4	—	—	—	—	—	—	—	
6D	7.9	1.23		10.9	11.6	9.8	10.2	9.8	10.1	—	—	—	—	—	—	—	
ϕ				0.160		0.128		0.119			0.108		0.160		0.137		
n				2.24		2.68		2.98			1.84		2.10		2.29		

^a Q_{PR} taken as Q_{PR} + Q_{SR} since Roxborough standpipes are very near Roxborough P. S. ^b Runs used to calculate ϕ , n and Σh .

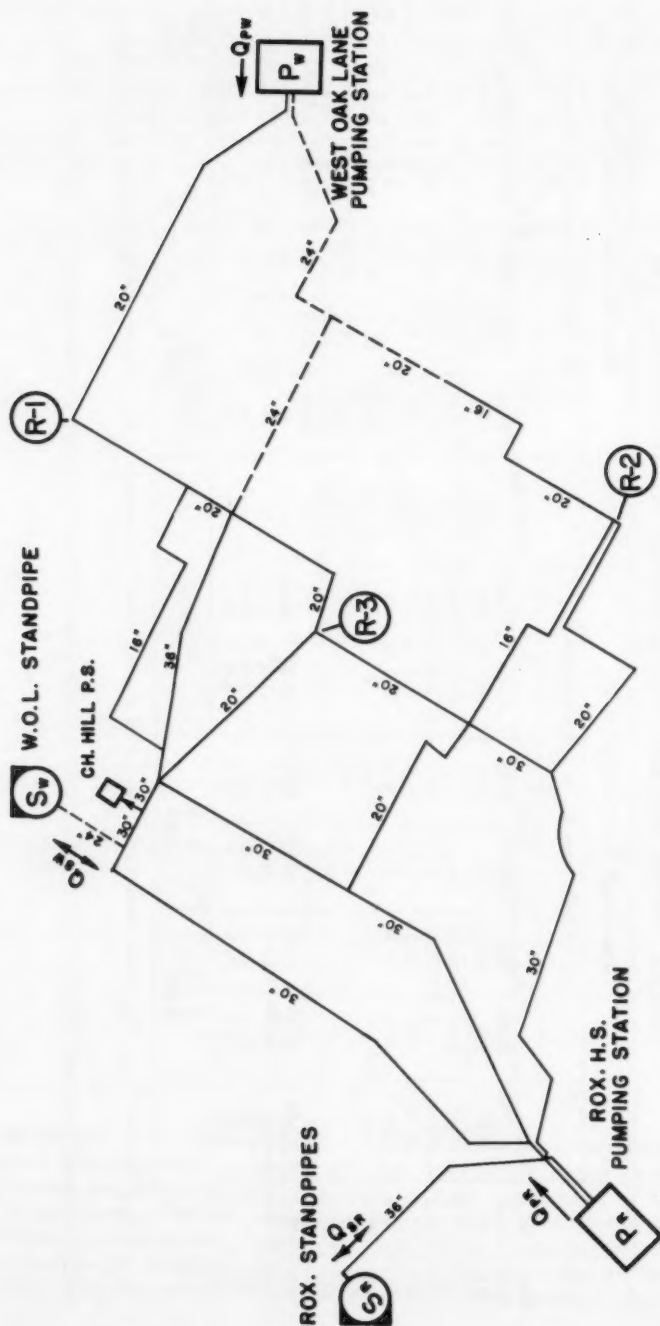


FIG. 7.—ROXBOROUGH HIGH SERVICE DISTRICT, ARTERIAL PIPING NETWORK FOR STUDY 4-5-6, NOT TO SCALE, AIRLINE DISTANCE FROM P_W TO P_R = 6.0 MILES

analyses were made, the twelve complete runs could have been reduced to about three or four, the remainder calculated and perhaps one or two quick runs made to check the results. If the Roxborough standpipes were located at the same site as the West Oak Lane standpipes, appraisal of the system characteristics would have been straightforward. Had the constant loading assigned the Chestnut Hill P.S. been about 20% instead of only 6% of the average demands, there would have been an obvious deviation between the "measured" and "calculated" head losses.

In the preceding examples the characteristics of only sample locations have been offered. Similar results can be obtained for practically any location in a network operating under proportional loading conditions. Naturally the corresponding equations representing different reference points will each have differing individual values of ϕ and n .

The number of composited local grid loads used in the analyses of the preceding examples are given in the following table:

District	Fig.	Number of Local Loads
Belmont Gravity	3	31
Torresdale H.S.	5	34
Belmont H.S.	6A	21
Chestnut Hill H.S.	6B	13
Roxborough H.S.	7	12

An unusually small number of loads were deliberately employed in the Roxborough H.S. study because of the complexity of the system.

CONCLUSIONS

Generalized network head loss characteristics have been demonstrated under the limitation of proportional loading. The use of proportional loads is an assumption usually incorporated in network design analyses. It appears that in general, near-proportional loading probably occurs in predominantly residential districts having small industrial or other specialized types of loads.

With proportional loads the percentage distribution of flows in individual pipes in a balanced network is constant irrespective of the magnitude of the total demand. This contention has been demonstrated by means of two elementary examples and inferred from the results of analyses for a more complex network where the sendout rate equalled the total demand. A generalized characteristic was deduced from this specific case for the more inclusive situation involving equalizing storage.

It appears safe to conclude that with systems which have, (or reasonably can be assumed to have) constant proportional demands, Eq. 2 is potentially a valuable calculating tool for determining system losses, particularly with equalizing storage. From the analyses for only two sample rates the head losses for other rates and/or supply source combinations can be speedily calculated. The computations for the successive hourly balancing of pump-network-storage functions can now be performed much more simply using Eq. 2, particularly with a digital computer, assuming of course that the results would lead to more significant or useful answers than can be obtained with an abbreviated procedure.⁽⁵⁾ The use of generalized characteristics would vastly simplify the calculations required in an economic evaluation recently proposed by Cole^(1-b) of different degrees of equalizing storage. The

potential value of generalized characteristics should not be under-rated. The engineer who must contract for computing services should now be able to reduce the time and cost of analyses. Using characteristics determined from field data it is possible to calculate losses for future expected demands with the existing system network to determine the maximum loading that can be sustained before system improvements become mandatory, without recourse to detailed analyses via a special computing device. Lomax(1-b) has stated that "The possibilities of the digital computer and the analyzer have not been fully developed" for use in network studies. It is anticipated that the use of generalized characteristics will accelerate a more advanced exploitation of these devices. The designer usually knows in advance the desired magnitudes of the quantities included in Eq. 2. Is it not feasible to consider writing a design program for a digital computer which would permit direct determination of the best combination of pipes to satisfy prescribed conditions defined by means of Eq. 2?

Discussion has centered about water distribution problems, with a constant value of m of 1.85. The characteristics presented can also be applied in analyses of gas distribution networks, wherein head loss relationships are related to the square of the flow.

The reader who is seeking a detailed summary of considerations which should be investigated prior to the performance of a network analysis will find the articles by Kincaid⁽⁶⁾ of particular interest.

ACKNOWLEDGMENTS

The network analyzer design runs cited were performed by Messrs. J. V. Radziul and P. Celenza of the Water Distribution Design, Unit, Philadelphia Water Department, Philadelphia, Pa. Mr. R. E. Wenzinger, student-employee of the Unit, performed the special Torresdale H.S. system runs. The information set forth in this paper was developed while the author was the Department's Research Engineer.

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NOTATION

Q_d = total demand (or load, or consumption) of system.

Q_p = sendout (or output) from a pumping station or from ground storage by gravity.

Q_s = flow into or out of elevated storage,
= $Q_p - Q_d$ (algebraic sum).

(Q_p/Q_d) = ratio of sendout to demand.

h = head loss in a single pipe or series of pipes forming part of a network loop.

Σh_1 = head loss from a sendout point to a given point in a network for a system without elevated storage.

Σh = same, but with elevated storage.

m = exponent in the relation for pipe flow resistance: $h \propto Q^m$.

K_1 = generalized coefficient for a system without elevated storage.

ϕ = generalized coefficient for a system with elevated storage. (when $Q_p = Q_d$, $\phi = K_1$).

n = generalized exponent of ratio (Q_p/Q_d) for case with elevated storage.



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FLOODS OF THE FLORIDA EVERGLADES^a

Discussion by Leo L. Burnet

LEO L. BURNET,¹—While Florida has the longest recorded history of any part of the United States, dating back to the early part of the 16th century, the interior of Florida, particularly the southern portion, has been one of the last areas to develop or even be explored. Several centuries went by before anything was known about the interior. The first known printed map of Florida, made about 1587, indicates the general shape of the peninsula with numerous geographic names dotting the coastline but shows nothing of the interior. The only name now recognizable is that of Cape Canaveral and is shown as "C. de Canareal." A map of Florida made in 1765 for the London Magazine, entitled "A New and Accurate Map of East and West Florida drawn from the best authorities," shows nothing of the interior except a maze of huge interconnecting waterways bearing little resemblance of conditions as they are known today. In 1837, one John Lee Williams prepared a map of Florida but "refused to show Lake Okeechobee - - - - because he doubted its existence." In fact, it was not until well in the middle of the 19th century, during the Indian wars, before white men learned much about the interior. Various tribes of Indians were entirely at home in the interior but not the white men. They didn't relish the thought of losing their heads. A few quotations from the history of Florida published by the State Department of Agriculture might be of interest:

"After his escape, Coacoochee rallied the Seminole chiefs and was largely responsible for the continuance of the war. He was one of the leaders at the Battle of Okeechobee on Christmas Day, 1839, and took part in almost every other encounter of any importance. He was only responsible for the lone comic incident of the war. On one occasion he attacked a theatrical troupe near St. Augustine and took their wardrobe. Soon afterward he was invited to a council, which he attended in Shakespearean costume.

"The war dragged on year after year. The Army even sent to Cuba for bloodhounds to track down the Indians, but they proved to be of little use. Gradually, however, the Seminoles were forced southward into the Everglades, which were first explored by expeditions of soldiers, sailors, and marines hunting Indians.

"The capture of Coacoochee in the summer of 1841 marked the beginning of the end. When he consented to emigrate with his band, other chiefs began to surrender. By August 1842, all the chiefs except Sam Jones and Billy Bowlegs had surrendered or been captured. Rather than continue the expensive war, the government assigned them a reservation in the Lake Okeechobee-Everglades area. On August 14, 1842, the Army announced that the fighting had ended.

^a June, 1959, by Paul Mayer.

¹ Ass't. Chf., Engrg. Div. U. S. Army Engr. Dist., Jacksonville Corps of Engrs., Jacksonville, Fla.

"The Seminole War greatly retarded Florida's growth. But it also did much to speed its future development. At its start, the interior of the peninsula was practically unknown - so much so that John Lee Williams, who had lived in Florida since 1821, refused to show Lake Okeechobee on a map he published in 1837 because he doubted its existence. The Army was a great exploring expedition that blazed trails and mapped the country. And many of the numerous forts and supply depots built by the troops later became towns."

* * * *

"Pioneers in the southern part of the peninsula were neighbors of those Seminoles who had refused to emigrate. The Indians lived quietly for a few years and then began to molest the whites. United States soldiers stationed at Fort Brooke, Fort Myers, Fort Lauderdale, and Fort Dallas (Miami) gave some protection to the settlers and, at the same time, made topographical surveys of the country.

"A military surveying party touched off another Indian war in December 1855, by destroying Billy Bowleg's corn and pumpkin patch just "to see old Billy cut up." The frontiersmen were in a panic and "forted up" behind palisades, but there were no extensive military operations. The government offered large rewards for captured Indians - \$500 for a warrior, \$250 for a squaw, and \$100 for a child. By 1858 most of the Seminoles, including Billy Bowlegs, had been rounded up and shipped west. A few however, still hid out in the Everglades. These events had no effect on the older parts of the state, which continued to develop along the lines begun in territorial days."

The southern part of the peninsula did not continue or even start to develop until considerably later. South Florida is unbelievably flat. To draw a profile clear across the state from west to east passing through Lake Okeechobee to a scale which would make it about 4 feet wide, all one has to do is to take a steel straight edge and strike a horizontal line, as the variation in elevation is so slight that it would amount to less than the thickness of a lead pencil line. The Everglades, an area of about 40 miles wide and extending about 100 miles directly south of Lake Okeechobee, is still flatter with absolutely no perceptible slope. For this 100-mile length the variation in elevation is only about 16 feet from north to south with virtually no variation from east to west. This north to south slope, if it can be called a slope, amounts to actually less than two inches per mile. Within this area there are no ridges, no hills nor valleys, and hence no defined water courses, since nature had no way of making them over this flat terrain. Rain water simply falls, stacks up and stays there with no discernible flow. It does, of course, move in a sheet slowly to the south but ever so slowly. Most of it gradually disappears through evaporation, transpiration, or seepage.

The lack of rivers or defined water courses is probably the primary reason why the interior of the Everglades remained a mystery to the white man for many centuries. Early explorers depended on rivers as a means of travel. Likewise, early settlers depended on rivers as a means of transportation, not only for themselves but for materials and supplies. The Everglades, entirely lacking of this means of transportation, remained undeveloped and largely unexplored until well into the 20th century. Here was a vast area of rich fertile lands with ideal climatic conditions which simply stayed as it was, inhabited only by the Indians.

Early in the 20th century, agriculture began to develop around the southern fringes of Lake Okeechobee. The big problem was the lack of drainage. The

land was so flat that rain waters simply wouldn't drain away. Eventually, four long canals were dug from Lake Okeechobee to the ocean, all taking a southeasterly direction. Agricultural development increased. These canals, however, did not accomplish their intended purpose, for the simple reason that you can't make water flow, canal or no canal, without a slope. Crops suffered severe flood damage. The answer lies in the methods used in the Central and Southern Florida Flood Control Project now in the process of construction. This project is unique in that it includes huge pumping stations to create an artificial slope in the main arterial canals. Most of these stations have been completed and are in operation. They have, indeed, been the answer to the problem.

The first part of the paper discusses the importance of the study of the history of the United States. It is argued that the study of the history of the United States is essential for a full understanding of the country and its people. The second part of the paper discusses the importance of the study of the history of the United States. It is argued that the study of the history of the United States is essential for a full understanding of the country and its people.

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ENGINEERING APPRAISAL OF HYDROLOGIC DATA^a

Discussion by Raphael G. Kazmann

RAPHAEL G. KAZMANN,¹ F. ASCE.—The classification of hydrologic data into three groups rather than two appears to be questionable. It is agreed that "basic" data and "analyzed" data as set forth in the paragraphs dealing with definitions are sound categories. The third category "interpretive" data is misleading and inaccurate if the example given in the report correctly depicts the Committee's views. As an example of "interpretive data" the Committee uses the phrase, "estimated yield of an aquifer."

Let us stipulate that an aquifer does not generate water. It merely stores and transmits water that has been derived, directly or indirectly, from the atmosphere. Consequently no interpretation of exclusively "basic" or "analyzed" data can possibly give a useful "estimated yield" figure because the basic data, while necessary, is insufficient for the purpose, as the following paragraphs elucidate:

It is possible to obtain a figure on rejected recharge. That is, if no one is using the aquifer as a source of supply, measurements and tests can be made which will tell us how much water the aquifer, being full, is rejecting.

But the concept of "yield" carries the connotation of "use." And the idea of use carries with it the idea that money will be spent in order to make the water available for use. The capital investment will depend primarily upon the estimated life of the water-using project, the desired rate of offtake, and the length of time in which it is desired to recover the investment. None of these variables is dependent upon the measurements made by hydrologists. Thus no mathematical manipulations of the results of physical measurements will enable engineers or scientists to arrive at a useful "estimated yield" figure.

This is not to say that measurements of rainfall, runoff, evaporation, permeability, thickness of aquifer, specific yield, etc., are valueless. These physical measurements are the indispensable raw materials upon which the engineering analysis will be based. They are necessary but not sufficient. But they will not result in an "estimated yield" figure unless economic requirements are superimposed upon them. Consequently any attempt to include "estimated yield" as a part of purely hydrologic interpretation is unsound and will result in needless and valueless effort.

The same remarks apply to studies of "safe" yields of aquifers (p. 13). The "safe yield" of any aquifer is limited by the amount of money you can afford to spend to recharge it and the quantity of water that you are legally entitled to use for that purpose. No matter how many observation wells are measured, no matter how many aquifer-performance tests are made, no matter how many

^a July, 1959, by the Task Group of Hydrologic Data of the Committee on Hydrology of the Hydraulics Div.

¹ Cons. Engr., Stuttgart, Ark.

test wells are put down, the observed data will not be sufficient to enable prediction of yield. The fundamental economic-engineering postulate of how much money will, and can, be spent determines the yield from any given physical matrix.

It is strongly suggested that the Committee Report be modified to exclude the sentence, p. 3 under "Interpretive data," "An example is the estimated yield of an aquifer" and substitute another example, if the Committee has one in mind. If no other example is forthcoming, "Interpretive data" should be removed as a category and the report modified appropriately.

It is suggested that, on p. 13, the sentence, "As water uses approach or exceed safe yields studies of limiting factors will become more detailed and data will become more important" be omitted.

On p. 15, item number 3) should be omitted in accordance with the preceding paragraphs, and the word "interpretive" should be omitted unless examples of "interpretive data" other than estimated yield of an aquifer, can be brought forward.

DESIGN METHODS FOR FLOW IN ROUGH CONDUITS^a

Discussion by F. V. A. Engel, P. Ackers, Nicholas Bilonok and
John A. Roberson

F. V. A. ENGEL.¹—The importance of Professor Morris's investigation lies in the fact that he directs once more our attention to a badly neglected subject, namely to transport phenomena of the flow of water, one of the most important elements serving mankind.

The present situation regarding our knowledge of friction phenomena is unsatisfactory, as most test results on friction date back to a period before Prandtl (1933) advanced his basic concept of turbulence. Prandtl's arguments were mainly demonstrated on the basis of Nikuradse's experiments on flow in rough and smooth pipe lines. No recent extensive investigations, extensive in the meaning of the complete work by Nikuradse, came to the knowledge of the writer. Most of the experimental work during the last twenty five years was rather limited in scope.

The new roughness concept by Professor Morris deserves a careful study. However, "refuting the Colebrook equation" should be clearly qualified. The author should indicate how far-reaching his statement is, as it appears rather unlikely that the results by Colebrook were completely unacceptable.

Furthermore, it is difficult to understand the significance of equation (8) presented at the beginning of the paper interpreting the regime of normal turbulent flow in accordance with Figs. 7 to 9. It should be emphasized that the Prandtl equation is in general agreement with Nikuradse's experimental results on rough pipes.

Comparing the equations by Prandtl and by Morris will result in pertinent queries.

Prandtl equation:

$$1/\sqrt{f} = 2 \log r/e + 1.74$$

Characteristic terms:

$$r/e$$

e = radial height of
roughness elements.

Morris equation:

$$1/\sqrt{f} = 2 \log r/L + 1.75$$

$$r/L$$

L = longitudinal spacing
of roughness elements.

(Note, the terminology in this discussion is in line with previous publications and differs from some of the terms used by Professor Morris.)

As already stated, the Prandtl equation describes very well the roughness pattern used in Nikuradse's experiments. What is the significance of Professor Morris's equation? A few questions arise, if two 'identical forms' of

^a July, 1959, by Henry W. Morris.

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equations describe the same phenomenon of roughness influence differing only in one of their terms, namely replacing e by L . Referring to both the Prandtl and Morris equations:

- a) can they both be correct?
- b) does neither one nor the other describe the phenomenon completely?
- c) are they complimentary?
- d) are there limited ranges in which either one or the other is applicable?
- e) are there ranges in which two independent parameters are required?

Perhaps Professor Morris would elucidate those points, which have been partly dealt with in both his papers, but the above analysis appears to warrant further explanation. Further comments are required, in particular with reference to the experimental evidence given in the following discussion.

Phenomena of roughness influences may be interpreted by analysing its meaning with reference to a typical example of uniquely defined roughness elements, namely orifice plates in series. (Uniquely defined with respect to relative height of roughness elements and also relative spacing of roughness elements.) There are three extensive investigations dealing with this particular pattern of roughness, namely by H. Möbius (1940),⁽¹⁾ W. Nunner (1956),⁽²⁾ and R. Koch (1958).⁽³⁾

The first task will be to determine the limit between 'small' area ratios and 'large' area ratios, of which the latter show the performance of a 'normal' roughness element.

Two distinct mechanisms of orifices in series as 'roughness' elements should be considered. For very large area ratios the type of wake interference flow, as dealt with by Morris occurs. The height of the element is small and its 'crest' is still close to the wall of the conduit. Eddies are shed from the edges of the orifice into the main stream, directed towards the centre line of the conduit. The second type of flow for 'small' area ratio orifices is acting in the opposite direction as eddies shedding from the jet are travelling in the direction of the pipe wall. The jet issuing from the orifice results in high central velocities at the axis of the conduit mixing with the nearly stationary fluid across the main section of the pipe. The disintegration of the jet may take place over a distance between 5 to 8 pipe diameters indicated by 'recovery' of a considerable amount of static pressure. This process is related to dissipation in accordance with the Carnot impact relation.

The Prandtl equation may be used as an approximate guide to determine the admissible relative height expressed as a limiting area ratio. Of course, the Prandtl equation could be modified by including correction terms so it may be applicable to smaller area ratios. However, there is no reason to do this in view of the arguments presented in the previous paragraph.

One of the most important investigations is a thesis by H. Möbius.⁽¹⁾ His Fig. 20 is partly reproduced in Fig. A presenting the friction coefficient related to the ratio of the spacing of the orifices in series to the height of the roughness element, i.e. the difference between the pipe and the orifice radii. From Fig. A it becomes evident that the reciprocal of the relative roughness represents a parameter of a set of curves. In the range of L/e values between 7 and 8 peak values occur. $L/e = 8$ indicates a demarcation line which distinguishes two different types of energy dissipation by the roughness elements. These types of dissipation are, however, not yet those which refer to the dissipation mechanism of 'small' area ratios which lie outside the range of validity of the general Prandtl equation.

In diagram, Fig. A, two series of tests by Nunner and Koch have been plotted. The Nunner tests refer to rings of 4 mm. height inserted in a 50 mm. pipe line; the cross section of the rings were semi-circular. They were evenly spaced in the pipe line and the total length was covered by 12, 24, 48 and 122 rings, respectively. The range of Reynolds numbers for turbulent flow conditions covers approximately 2,000 to 100,000. The resistance values were nearly constant, only for the two last test points there was a slight rise in the resistance value with increasing Reynolds number. The test points fit in fairly well with the general trend of the Möbius diagram, clearly indicating a distinction between L/e - values smaller and larger than 8.

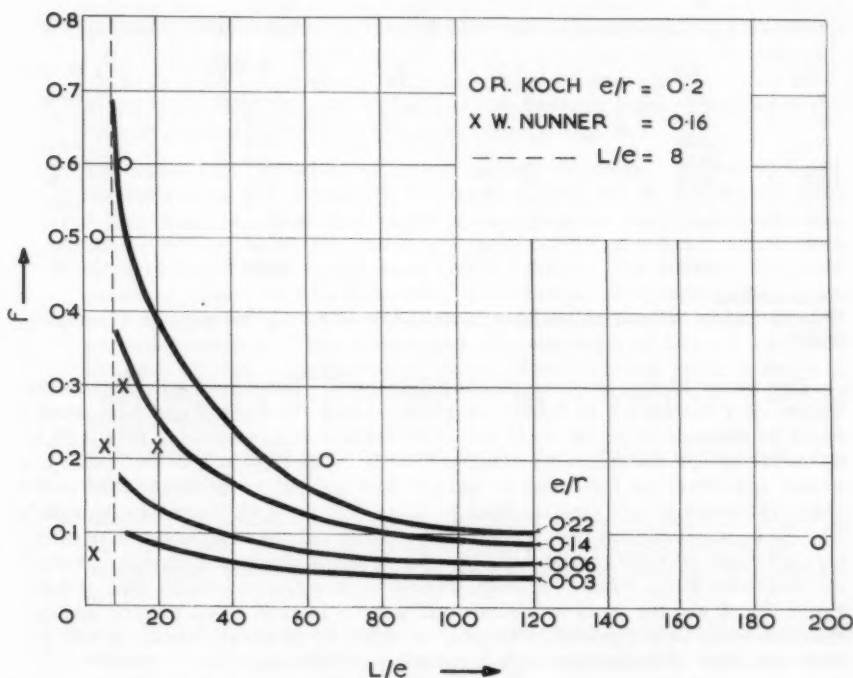


FIG. A.

Four test points by Koch of differently spaced square-edged orifices in series of an area ratio of 0.64, corresponding $e/r = 0.2$, fit in fairly well with the Möbius curve for the ratio of 0.22. However, for the reasons given above this area ratio should already be considered as a limiting area ratio. It is outside the scope of the Prandtl equation, as becomes evident from the following figures:

$r/e = 5$	$f = 0.09$
$1/\sqrt{f} =$	3.333
$-2 \log 5 =$	<u>-1.398</u>
	1.935

This value is greater than the figure 1.74, as specified by Prandtl. Until further evidence is available area ratios of 0.70 should not be considered as roughness elements in pipe lines of 50 mm. diameter. However, in pipe lines of other diameter due to scale effects the limiting area ratio may differ in its numerical value.

The meaning of the demarcation line of Fig. A should be analysed as it may result in some qualification of both diagrams, i.e. Figs. 7 and 8. in Professor Morris's paper. Table I has been prepared to enable a comparison between the Möbius diagram and the Morris diagrams, which otherwise would not be possible in view of the different roughness definitions.

TABLE I.—FRICTION COEFFICIENTS RELATED TO ROUGHNESS PARAMETERS.

L/e	e/r	r/e	r/L	f Möbius	f Morris.
1	0.1	10	10	(0.032)*	0.07
8	0.125	8	1	0.26	0.31
8	0.06	16.7	2	0.19	0.2
10	0.1	10	1	0.24	0.31
12.5	0.04	25	2	0.12	0.19
16.7	0.06	16.7	1	0.15	0.31
16.6	0.03	33.3	2	0.08	0.19
33.3	0.03	33.3	1	0.06	0.31
50	0.02	50	1	(0.05)*	0.31

* extrapolated

(Note the values are only approximate, which is sufficient for the purpose of comparison.)

The scope of the table is somewhat limited. The range of numbers for values of r/L from 2 to 5,000, as given in both the Figs. 7 and 8 by Morris could be obtained either by very small values of e/r , say smaller than 0.03, or by values of L/e considerably smaller than 8. The first row of the Table I is rather uncertain as the trend of curves has not yet been established in the range of values of L/e smaller than 8. In accordance with Professor Morris's Fig. 10 hyper-turbulent flow should prevail under those conditions. It is of interest to note that the lowest value for the relative roughness spacing given in the diagrams Figs. 7 and 8 is unity, resulting in a friction coefficient of 0.31. From Fig. A and the tests by Nunner and Koch it is known that the friction coefficient could be considerably larger. It would be of interest to know why the diagrams have been limited to parameter values of unity.

One feature which becomes evident from Table I is that r/L does not uniquely define the resistance coefficient f . For $r/L = 1$ covering a range of values of r/e from 8 to 33.3 results, in accordance with Möbius, in a decrease of the resistance coefficient from approximately 0.26 to 0.06. The Morris conception would result in a single value of $f = 0.31$. A similar characteristic is disclosed by the few examples for $r/L = 2$. The divergence in Fig. A of the curves is remarkable when they approach the limit of 8 from larger values of L/e . This means that r/L is not sufficient to define the resistance coefficient and that r/e or another comparable parameter is required to determine the coefficient f .

There is another range of the friction characteristic which does not appear to be quite in line with the results by Nunner and Koch. Fig. 10 shows a comparatively short range of Reynolds numbers between the limiting curve for the smooth surface and the onset of 'normal turbulent flow' when the friction

coefficient becomes constant. Fig. 8, for sharp-edged strip roughness, presents in contradistinction a rather extended intermediate or transition regime. The tests by Nunner and Koch show in general an abrupt change at low Reynolds numbers, say between 1,000 and 3,000, after which the friction coefficient becomes in most cases nearly constant, as shown in the normal flow range in the Morris diagrams. Some further elucidation of this point is requested.

Summarising the previous remarks the writer would like to ask Professor Morris to comment on the following issues:

- 1) Values of r/L in the range from 10 to 5,000 could only be obtained by very small values of e/r , say between 0.02 for a limited range of values of L/e , say smaller than 10 and preferably considerably smaller than unity. The latter condition appears to be in contradiction to the requirements of wake interference, as it should result into quasi-smooth flow. Would the author, therefore, explain the various feasible ranges of the complete set of parameter values as given in Fig. 8.
- 2) Two resistance characteristics with opposite tendencies appear to exist; one for values of L/e smaller than 8 and another for values larger than 8 (see Fig. A). How is it possible to reconcile those two distinctive characteristics with the parameter r/L ? This parameter cannot disclose to which range of values of L/e it belongs. The author's complete range of values of r/L from unity to 5,000 with reference to Fig. 8 can only be covered by correlated and comprehensive ranges of L/e and e/r parameter ratios. This means that the peak value of $L/e = 8$ should be included in this comprehensive range. How can this be achieved? It means that some of the parameter values, as given in Fig. 8, are not single-valued.
- 3) Furthermore, values of e/r should be limited. Which limit had the author in mind?
- 4) How can the transition ranges in Fig. 8 for Reynolds numbers between, say 1,000 and 100,000 be brought in line with Nunner's and Koch's investigations which show in this range a practically constant friction factor?

At the end of the paper a statement is made "that the concepts correlated data obtained from many different sources." As reference (16) is not precisely given and may not be available in print the author could add much to his very important paper if he would present in the Closure a table indicating the ranges covered by the investigations which are in line with his statements. This table may include the various types of roughness flows and ranges covered with respect to Reynolds numbers and values of r/L and e/r .

Professor Morris rightly points out that "further study directed to the specific end of elucidating and refining the concepts" are very desirable. To the best of the writer's knowledge no extensive and reliable investigations have been made since Nikuradse's work about thirty years ago. If any new work is contemplated the experiments should be carefully designed and based on the latest advance of our knowledge in the field of fluid dynamics. Some requirements are, of course, rather stringent and not easy to fulfil. Often the results of the experimental investigations are inconclusive due to insufficient preliminary preparation of the experiments, in particular with regard to planned

layout. Badly designed entrance conditions, too short length of the rough conduit section (100 pipe diameters length of the rough surface may be required in case of turbulent flow and over 300 pipe diameters for laminar range) and the failure to ensure iso-thermal conditions have given contradictory or inadequate results.

If and when such investigations are planned and executed another important problem should be settled, i.e. the correlation of the velocity profiles and the essential roughness parameters covering an extensive range of Reynolds numbers. This, of course, will give a better insight in the mechanism of roughness. The question is, whether the velocity profile under all conditions of roughness (illustrated in Fig. A) is sufficient to explain fully the significance of the roughness and turbulence problems. Some recent investigations⁽⁵⁾ on the influence of roughness and various roughness patterns on the discharge characteristic of measuring orifice plates may lead to the conclusion that the velocity profile may not be a sufficient characteristic in case of excessive roughness. At least the results of this investigation appear to be considerably above the values, which were related⁽⁴⁾ to a change of the frictional coefficient f in the range from 0.015 to 0.06. In view of the many important and still unsolved problems it is hoped that Professor Morris's paper will stimulate some systematic and basic research on friction coefficient in conduits, which is certainly overdue.

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P. ACKERS.¹—This paper forms an important continuation of his previous paper⁽¹⁾ on the subject: it will be especially valuable in estimating head losses in pipes with unusual forms or combinations of roughness, for which experimental data are not readily available. In the writer's view, these important ideas would be more readily accepted if additional evidence directly in support of the proposed equations, as opposed to pre-existing ones, could have been quoted. Much of the experimental information on commercial surfaces mentioned in this latest paper has not been compared directly with the new equations to the writer's knowledge, but is referred to (somewhat obliquely) as being contrary to previously accepted equations.

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Morris's earlier paper gave a carefully reasoned account of the influence of the form and spacing of the turbulence - producing elements on the resistance function, leading him to describe three basic forms of turbulence in rough conduits: Isolated roughness, wake-interference, and skimming flow. Unfortunately, his present paper is introduced by some "below the belt" criticism of the brilliant work of Colebrook and White some twenty years ago on fluid resistance, and, in the writer's opinion, he does his own field of study a disservice by thus discrediting one of the mile-stones in its progress. The writer freely concedes that the Colebrook-White transition equation is but one stage in the quest for knowledge about fluid resistance, and, although it was of very great significance, it must in time be discarded in favor of even better equations. The author's recommendations undoubtedly point the way ahead, but nevertheless his adverse comments on the work of Colebrook and White call for reply, if only to keep the record straight.

Most people who have studied the modern concepts of fluid resistance are aware of the meaning, and limitations, of the "equivalent sand roughness" used in the Colebrook-White equation. It is the linear measure of roughness which would yield the same friction factor as the equivalent Nikuradse sand size, under conditions of fully-developed rough-turbulent flow. Morris's suggestion that, to be acceptable, the Colebrook-White function should show the same form of transition as Nikuradse's sand is contrary to the whole purpose of a commercial-pipe resistance equation, and merely confuses the issue. Colebrook's Fig. 1⁽³⁾ clearly demonstrates that equivalence is confined to the square-law region. He fully recognised too that the equation he and White derived was applicable only to those types of surface which give a descending transition, and that where the roughness elements were so close as to give interference between their wakes, a quite different function was required. "Any attempt to express mathematically the transition function for uniform sand-roughness is rendered difficult owing to the fact that the turbulent motion in the wake behind the grains is complicated by mutual interference" wrote Colebrook, in anticipation of some of the ideas propounded by the present author. Experiments on pipes artificially roughened with mixed sizes of sand were also described:⁽²⁾ those with a uniform application of small grains followed the Nikuradse "wake-interference" transition, those with a background of small grains and a superposed pattern of larger isolated grains gave a generally horizontal transition, whilst isolated grains by themselves gave the familiar descending transition. Then, in ref. (3), Colebrook analysed data on many commercial pipes, showing that they gave a similar transition to the isolated grains, and commented thus: "It is seen that although some of the pipes do not agree very closely with the mean curves, some having too rapid a transition and other too slow, there appears to be sufficient evidence to justify the adoption of the given mean transition laws together with the mean k -values." It is remarkable, in fact, how many commercial surfaces do produce the "isolated roughness" type of transition.

The author quotes certain experimental data as refuting the Colebrook-White function, referring for example to Burke's experiments on a very large penstock.⁽⁴⁾ However, these tended to confirm the generality of the equation under criticism, rather than refute it. Referring to "the exceptionally smooth surface produced by enameling the inside of the pipe," Burke wrote, "It can be concluded that the present data have provided a substantial experimental verification for the extrapolation of the von Karman or Nikuradse smooth-pipe equation to values of the Reynolds number up to approximately 3.8×10^7 from

previously published high values of about 3×10^6 ." There is no conflict with the Colebrook function there! "Data for the 51 inch pipe-line apparently differ from those for the 123 inch diameter pipe, departing from the smooth-pipe curve and showing a distinct tendency to assure a constant f -value." Burke's data for the 51 inch pipe-line result in $k \approx 0.0002$ ft, which is entirely consistent with data on smaller pipes with a similar surface finish. Morris, referring to his own experimental studies of 18 inch, 24 inch and 36 inch concrete pipes, says "Certain tests have yielded systematic variations in empirical values of equivalent sand roughness for a given surface, proving it to be unreliable Various data are available reputing the Colebrook equation implication that friction factors for a given surface type always decrease with increasing diameter." Yet when we study the concrete pipe experiments we find that the pipe-lines referred to were not identical. The author himself says (1) "The joints in the 18 inch diameter pipe were considerably smoother and fitted more snugly than on the pipes having diameters of 24 inches and 36 inches," so that any differences in their apparent roughnesses is almost certainly real, and does not necessarily fault the Colebrook-White equation. In fact, these concrete pipe data follow the transition function quite closely. The writer has recently tested 12 in. diameter spun concrete pipes with good joints and found that their equivalent sand roughness was 0.00013 ft. data for a carefully-finished concrete tunnel of 45 ft. diameter has been published recently, (5) showing its roughness to be 0.00019 ft. Surely this is admirable confirmation of the generality of the Colebrook-White equation for surfaces giving an isolated-roughness type of transition. Campbell and Brebner (6) have given us further supporting evidence as a result of experiments on 2 in. to 6 in. diameter aluminium pipes: "This relationship between these results (relative roughnesses for three diameters of pipe) is as one would expect, since the surface finish on aluminium pipes remains sensibly constant whatever the diameter, and thus with increase in diameter the relative roughness should vary inversely with the diameter."

It is worth mentioning too that over twenty years ago Colebrook and White (2) gave some consideration to the influence of the shape of roughness elements on the relationship of the equivalent sand roughness to their physical dimension, and also demonstrated that the spacing of the protuberances entered the relationship, an idea which Morris has now carried very much further. The main advantage of the author's approach to fluid friction, in comparison with previous methods, is that it permits an estimate of friction in a pipe or channel whose roughness can be defined by physical measurements, even if the roughness is of such a form that a "non-Colebrook" transition will occur. However, it seems that in the design of the majority of commercial pipe-lines, the engineer may still have to rely on semi-empirical estimates of friction until such time as he can reliably predict the probable values of the height, longitudinal and peripheral spacing, and form, of the protuberances which might be expected with any proposed method of construction. It is to be hoped that such data will be collected whenever possible, and comparisons then made between estimated and experimental friction losses in commercial conduits.

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NICHOLAS BILONOK.¹—This paper has presented curves and methods for determining friction factors for turbulent flow in closed conduits and open channels. The presentation is directed to better understanding of the physical regimes of turbulent flow and of pipe friction.

The characteristics of the five types of turbulent flow are well defined, and for each type presented equation of friction factors, in very complicated form. The various factors entering into the problem based on the "assumptions" and coefficient "perhaps range from. . . to about," "probably reasonable and conservative" . . . appears hardly adequate.

The first question that will occur to the designer: How much does the loss of head computed as suggested in this paper differ from that computed by other formulas. Second: In the event that this substationally rational method in nature should be adapted, what simple and accurate method to find the type of roughness element and coefficient C_D , when all type of roughness elements from Fig. 5A can be found in the pipe under consideration?

The main obstacle to the comprehensive solution lies in the complexity of the roughness pattern and in the resulting difficulty of defining and hence of measuring roughness. The average approximation may be very far from individual cases. We have to remember that some of complicated formulas are not necessarily less approximate than the simpler one.

The equivalent roughness element diameters k of Nikuradse⁽¹⁾ are used in theoretical investigations and evaluation the effect of roughness in pipes, depend on the ratio of the roughness size to the radius of the pipe. The main obstacle encountered here is the difference in units of roughness. Due to the simplifying assumption and uncertainties in relationship of variables, the statistical investigations failed to reveal the effective of value of Reynolds number in open channel flow,⁽⁴⁾ also recent investigation and measurements in the existing circular 51 in. and 123 in. in diameter pipe lines⁽⁷⁾ show some negative informations as to application of the Nikuradse roughness law. Results of the friction factor tests indicated that the care taken in fabricating and lining the pipes produce one major desired result the attainment of a highly favorable of the Manning n .

An analysis of the test data indicates that the surfaces concerned to conform to the rough pipe low of Nikuradse and that a change of constants or possible new low, might serve to express the roughness of surface that may be classified as "wavy."

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Experiments which have been performed with great care and under perfect conditions, as many investigators stated, frequently fall far from the values determined from other experiments. It appears, that there is danger in accepting any formula designed to give mean values.

One single formula or diagram can not express all condition of flow of fluids of any kind in closed conduits and open channels. It does not matter to change exponent at one or the other variable, or change constant number in mathematical expression, the main problem is how this formula corresponds to the real conditions of the flow. A distinct difference between closed conduit under pressure, is that the open channel depends upon the slope given their free surface and the conduit depends upon an external head for the production of flow. The analytical full treatment is more difficult of flow in open channel, because of the wide variation in the conditions and roughness coefficient must be determined for each type and shape of surface. In the nature of the lining, cross section may have an infinite variety of shapes and change from section to section. Under these circumstances, it is exceedingly difficult to derive a formula for flow that will be general in its application for pipes and open channels.

Formulas of Chezy, Kutter, Manning, Hazen-Williams are in form convenient for solving practical problems that are in general use. All these formulas are empirical applicable to the water flow only, and may be confidently adapted only if properly applied to experimental factors.

Definite progress has been made, but coordination between laboratory and field experiments, as well as between theory and designing office practice has not been accomplished.⁽¹⁸⁾

The physical law of liquid flow in the conduits is one, and can not be expressed numerous formulas. The most important is the correct form of equation based on the theory, and numerical coefficients performed from the tests, and direct measurements with great care in full size of pipe.

Most of the results of investigations have not been satisfactory because of their wide disagreement, which may be attributed to the absence of a standard basis for their correlation.

The detailed refinement for only one factor, f may create illusion that, by default the other variables are associated with a small margin of error. It is quite evident that there is much remaining to be investigated from the standpoint of theory and basic experiments, if a higher degree of precision in designing computations is to be achieved.

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JOHN A. ROBERSON,¹ M. ASCE.—The author has attempted to put the problem of the determination of flow resistance on a more rational basis than has existed in the past. For this attempt and for introducing concepts which deserve continued study, the author is to be commended. It is felt that the author's division of flow into various regimes of turbulence is necessary to pursue the complex problem of flow in rough conduits.

It is the writer's opinion, however, that verification of the author's hyper-turbulent flow theory is yet to be accomplished. One of the author's main conclusions from the hyper-turbulent flow theory was that "for sufficiently high Reynold's numbers, each transition function approaches the normal turbulent flow equation for which the friction factor depends solely on the relative roughness spacing." This conclusion was developed in a previous paper by the author⁽¹¹⁾ in which a key hypothesis was that wakes behind the individual roughness elements could be made similar (implied by stating that v/v_s values could be made similar) regardless of roughness height or shape. The reasoning leading to similarity of wakes followed from the author's concept that wakes may be altered because "the location of the separation point on the roughness elements can be adjusted by adjusting R_c ." To test the hypothesis one might take the extreme case of two different types of angular type roughnesses. Consider one type as cubical roughness elements and the other type as pyramidal roughness elements. It is generally agreed that the point of separation will not change for almost the entire range of R_c for either of these types. In other words, separation will occur along the edges of the elements. Moreover, there is no reason to believe that their wakes should be similar to the outset; consequently, there is no justification in this case for assuming that the wakes will be similar. Furthermore, it seems that by attempting to test the hypothesis on rounded type elements the conclusions arrived at would be less decisive due to the added complexity.

If the writer has misinterpreted the author's hypothesis concerning constancy of v/v_s which implies similarity of wakes, clarification of this part of the hyper-turbulent flow theory might result if the author would restate the hypothesis in a different manner. Although the author in his earlier paper presented data attempting to justify the conclusion that for normal turbulent flow " f " is a function of relative spacing only, it seems that the data contained too few examples to test adequately the influence of spacing alone as opposed to roughness form and height.

The writer concludes that the concepts, equations, and curves presented by the author certainly provide ideas for analysis and future development of the theory of flow resistance in rough conduits, but do not yet provide a design tool.

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DETERMINATION OF HYDROLOGIC FREQUENCY FACTOR^a

Discussion by L. L. Weiss

L. L. WEISS.¹—According to Chow,⁽¹⁾ the general formula for hydrologic frequency analysis is:

$$x/\bar{x} = 1 + C_v K \dots\dots\dots (1)$$

where x = the magnitude of a hydrologic event

\bar{x} = the arithmetic mean of x

C_v = the coefficient of variation of x

and K = the frequency factor for a given probability of occurrence.

The value of K at a given probability will, of course, depend on the distribution under consideration. In his article Chow shows how to determine values of K for a lognormal distribution by a graphical method and suggests its use as a practical measure.

Aside from the inherent difficulty of accurately reading the ratio from the constructed line, the suggested computed points for its construction are too close together for greatest accuracy at high and low probability points. In addition, after the ratio is determined, the K values must still be computed.

A somewhat different method for determining K has been used by the writer. When C_v has been given, it expresses K directly as a function of the probability. This may be preferable to either the author's nomogram or the writer's nomogram.⁽²⁾

The relationship is developed as follows:

$$\bar{x} = \exp(\bar{y} + \sigma_y^2/2) \dots\dots\dots (2)$$

where \bar{y} = the natural logarithm of x

\bar{y} = the arithmetic mean of y

and σ_y = the standard deviation of y .

substituting (2) in (1) and taking natural logarithms:

$$y - \bar{y} = \sigma_y^2/2 + \ln(1 + C_v K) \dots\dots\dots (3)$$

Expressing (3) in standard units t , gives:

^a July, 1959, by V. T. Chow.

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$$t = \sigma_y/2 + 1/\sigma_y [\ln(1 + C_v K)] \dots\dots\dots (4)$$

and this is the required relationship.

If we take the example given by the author of $C_v = .364$

Then from

$$\sigma_y^2 = \ln(C_v^2 + 1) \dots\dots\dots (5)$$

$$\sigma_y = .353$$

Which in (4) gives:

$$t = .1765 + 2.832 \ln(1 + .364 K) \dots\dots\dots (5)$$

and on solution for the probabilities chosen by Chow in his table 1, gives precisely the K values given by him.

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1. Chow, Ven Te, The log-probability law and its engineering applications, Proceedings, Amer. Soc. Civ. Eng., Separate 536, 1954.
2. Weiss, L. L., A nomogram for log-normal frequency analysis, Trans. Amer. Geophys. Union, v. 38, n. 1, pp. 33-37, Feb. 1957.

TRANSLATIONS OF FOREIGN LITERATURE ON HYDRAULICS^a

Discussion by John B. Herbich

JOHN B. HERBICH,¹ M. ASCE.—The following translations were made by the Hydraulics Division, Fritz Engineering Laboratory, Department of Civil Engineering of Lehigh University, and it is suggested that they be added to the list of translations:

- (1) Armanet, L. - "Genissiat (Power Station) Turbine Butterfly Valves." *La Houille Blanche*, pp. 199-219, 1950. Tr. by P. J. Colleville. Fritz Engineering Laboratory Translation No. T-1, 1953.
- (2) Krisam, F. - "Influence of Volute on Characteristic Curves of Centrifugal Pumps" (Der Einfluss der Leitvorrichtung auf die Kennlinien von Kreiselpumpen), *Zeitschrift des Vereines Deutscher Ingenieure*, Vol. 94, No. 11/12 April, 1952, pp. 319-322. Tr. by A. Ostapenko and J. B. Herbich. Fritz Engineering Laboratory Translation No. T-5, April 1959.

^a August, 1959, by The Task Force on List of Translations of the Committee on Hydromechanics of the Hydr. Div.

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